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WASTE WATER ASSIMILATION STUDY

of the South Nation River Watershed

Main Report

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WASTE WATER ASSIMILATION STUDY of the South Nation River Watershed

Main Report

January, 1993

Prepared by Gore and Storrie Limited (Ottawa)
for the South Nation River Conservation Authority
and the Ontario Ministry of the Environment

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EXECUTIVE SUMMARY

The South Nation River watershed has been studied for its potential to assimilate wastewater effluents from point sources at existing and anticipated future conditions. In the future, in addition to population increases in existing communities, other communities will be discharging waste from water pollution control facilities. There are two industries, Nestlé and Ault Foods Ltd., currently discharging waste and one additional industry will discharge in the future. Except for the two industries, all of the communities that have water pollution control plants are using conventional facultative lagoons that discharge on an annual or semi-annual basis. Lagoons are being contemplated for communities that will release their wastewater to the South Nation River in the future. The future planning period for this study was 20 years. Industries were consulted for their expected growth during this period. The populations of the communities 20 years from now were estimated from population studies for the region and planning documents prepared by the municipalities.

Allocation of assimilative capacity for future conditions was dependent on estimates of future populations, specifically, regional population for the year 2011. For the purpose of this study, future population estimates were generated from growth projections of individual municipalities. Sources for the municipal growth projections included reports from independent consultants and various Official Plans. The resultant regional future population estimate, for the year 2001, was approximately 37,000, which is almost twice the present regional population. This population estimate provided the bases for the waste allocation analysis for future conditions. Detailed assessment of this regional population estimate, and the implied spatial distribution of growth, were beyond the scope of this study. Since the assimilative capacity of the South Nation River is, in effect, a shared resource, the regional population growth, rather than the growth aspirations of individual municipalities, should be assessed.

The existing lagoons were characterized for their design features and performance based on historical data. There was only a limited database on treatment facility performance. Phosphorus removal through alum addition is being practised at a number of facilities. The performance of existing lagoons is typical of lagoons found in Ontario. Additional performance data from lagoons in Ontario were used to arrive at conservative lagoon effluent quality estimates. The industries have custom designed wastewater treatment facilities which have experienced some difficulty in recent years in meeting Certificate of Approval effluent specifications.

All available water quality and flow data for the streams in the watershed was analyzed. Water in the streams in the watershed is alkaline with a pH typically higher than 7.5. Water quality in the South Nation River system has been improving during the past 12 years as additional wastewater treatment capability has been installed. However, the background quality in the streams consistently remains above the Provincial Water Quality Guideline for phosphorus. Ammonia levels are also high. On an annual basis the major source of ammonia and phosphorus is nonpoint sources. Annual nitrogen and phosphorus loads contributed by nonpoint sources to the South Nation River are greater than 98 and 93%, respectively. For the period of point source discharge, however, the point source contribution constitutes more than 50% of the load in the river for ammonia, the most problematic substance. Furthermore at low flow conditions the load contribution from point sources will significantly increase as a percentage of the total load in the river.

Background concentrations of pollutants in the stream were estimated from the 75 percentile stream concentrations yielding a BOD_5 of 2.3 mg/L and total ammonia concentration of 0.18 mg/L were found.

With present background concentrations of phosphorus in the South Nation River system there is no assimilative capacity for phosphorus. New point sources will have to achieve low effluent total phosphorus (TP) concentrations along with concurrent reductions in non-point sources to return the rivers to a desirable state with respect to this pollutant. In existing lagoons, phosphorus reduction (usually with alum addition) has only been initiated recently.

The MOE policy currently requires that present and future TP loads remain at or below current loads and that TP concentrations of treated wastewater be below 1 mg/L. At present, Plantagenet, Winchester and the Ault Foods industry are at the limit of loads defined by their Certificates of Approval. Phosphorus loadings at future flow conditions are not expected to increase, and a reduction in current loadings is desirable.

The assimilative capacity of the streams in the system was analyzed at 7Q20 and 30Q20 flow conditions. A low frequency flow analysis was carried out to determine these flows at each of the water gauging stations during the spring and fall periods. A moving window approach was used to characterize the 7Q20 or 30Q20 hydrographs at each station.

From these hydrographs, the period during which the maximum volume was available for dilution at each station was determined. Of course, for each station, the period of maximum dilution had distinct starting and ending dates. By comparing these starting and ending dates for the period of maximum dilution at each station, three windows were defined: the period included between the earliest starting date and the latest ending date was defined as the maximum window (it had maximum duration); the period included between the latest starting date and earliest ending date was defined as the minimum window (minimum duration); the period included between the average starting date and average ending date was defined as the average window. Using the windows concept allowed greater utilization of the available river flow for wastewater assimilation and dilution.

The dilution volumes and stream flow rates available in each of the windows were determined for each station and the correlation between contributing drainage area and flow was estimated. These correlations were used to estimate flows at all existing and potential wastewater source locations and at other points in the watershed where stream confluence occurred. Fall 7Q20 flows were indirectly estimated based on the ratio of drainage area at the location of interest to the drainage area at Plantagenet. The 7Q20 flows at Plantagenet were estimated with statistical techniques. Scatter in the data at other stations was too large to allow the full implementation of the procedure used for the other flow conditions.

In addition to lagoons, extended aeration with nitrification and New Hamburg processes were considered as possible upgrades to existing treatment. Effluents from all processes were examined for new toxicity standards being contemplated by MOE. The lethality standards for effluent are un-ionized ammonia less than 0.1 mg/L and undissociated hydrogen sulphide at 0.02 mg/L. These standards apply to the effluent before it is discharged to the receiving water (in-pipe standards). To insure compliance with the sulphide standard, it will be necessary to install post-aeration cells for all lagoons. It is also likely that lagoon effluent will violate the ammonia standard, particularly for spring discharges which is the prime discharge period for lagoons.

Effluents for extended aeration or New Hamburg treatment are likely to be compliant with effluent lethality at all times.

Spring lagoon effluents were assumed to range between 1 and 3 mg/L for undissociated hydrogen sulphide. There is insufficient dilution capacity available at any location in the spring under either present or future conditions to reduce lagoon effluent undissociated hydrogen sulphide to PWQO stream values. It will be necessary to install a post aeration cell for a lagoon discharging in the spring to guarantee compliance with the in-stream hydrogen sulphide standard.

In this first comprehensive study of assimilation in the watershed, the conservative and practical constraint of approximately constant discharge rates for the effluents was imposed. Maximum dilution factors available in the spring and fall were insufficient at some locations to meet PWQO objectives for ammonia. Because of the alkaline nature of the waters and the high background concentrations of ammonia, ammonia was the limiting pollutant. Relocation of some effluent discharge points to locations where sufficient dilution capacity is available was necessary.

A spreadsheet model was developed to characterize the watershed. A modified Streeter-Phelps equation incorporating nitrogenous oxygen demand and sediment oxygen demand was used to characterize dissolved oxygen variation. BOD and ammonia were decayed according to first order relations. Sediment oxygen demand was assumed to be present only in the fall due to accumulation of deposited organics over the low flow summer period. During the spring it was assumed that the higher stream flows would scour deposited organics from the streams. The model incorporated waste flows, river and tributaries base flows as well as groundwater inflows at appropriate nodes. The spreadsheet model evaluated all streams to which point sources discharged and linked discharge windows for a more convenient automated solution.

Low flows in the fall as well as background dissolved concentrations and sediment oxygen demand resulted in PWQO dissolved oxygen violations without any waste sources discharging. Therefore the fall period was eliminated from consideration for waste discharge. The current and future conditions were optimized to meet the PWQO using the model. The solution approach was to maximize use of the assimilative capacity starting at the upstream end of the streams and working downstream.

During spring ammonia was found to be the pollutant limiting discharges. The results of the wastewater allocation optimizations are as follows.

- Small flows in McMartin Drain will force Williamsburg to install extended aeration treatment.

- Winchester, Maxville and Ault Foods Limited should relocate their effluent discharge points for all conditions. While Maxville and Ault Foods Limited must discharge further downstream into local receiving streams, Winchester must move its discharge point to the South Nation River.

- At present and future growth conditions under 30Q20 stream flows, all other communities are able to use lagoon treatment.

- At present conditions under 7Q20 stream flows, all other communities are able to use lagoon treatment.

- At future growth conditions, under 7Q20 stream flows, Williamsburg, Embrun and Moose Creek should upgrade to extended aeration treatment or better. All other communities are able to use lagoon treatment.

- Staged discharge schemes were developed for the discharge of effluents under all windows. As conditions changed from present to future growth conditions or from 30Q20 to (the lower) 7Q20 flows greater use of all discharge windows had to be made.

A mixing zone analysis was another possible limitation studied. The criterion was that any waste source had to maintain a 75% zone of passage outside of the effluent plume where the concentrations of pollutants were less than the PWQOs. The analysis was limited to ammonia and only conducted for spring discharge conditions because of uncertainty in fall travel time and flow estimations. The 75% zone of passage objective could be achieved everywhere on the South Nation River for both present and future 30Q20 conditions, and for the present 7Q20 conditions. It could not be achieved in the upper reaches of the South Nation River for future 7Q20 conditions. Most tributary streams could not achieve the 75% zone of passage under any flow condition.

A cursory evaluation of costs was conducted to compare the various treatment options. The New Hamburg and extended aeration processes were found to have similar capital costs.

Major recommendations from this study include a more extensive monitoring of water quality in the watershed and a stochastic analysis to better define frequency and duration of deleterious stream conditions. Nonpoint pollution sources must be evaluated and corrective measures assessed for their potential to reduce background concentrations of nutrients and other contaminants.

Real time control of effluent discharges with variable rate discharges linked to streamflows offers the potential to reduce the treatment upgrade requirements or discharge point relocation measures.

Wastewater assimilative capacity is one of the factors which can affect or limit growth. Water allocation is another which should be considered. There is a requirement to develop a clear guideline on wastewater and water allocation for a watershed. Besides water and wastewater quantities and costs, the secondary effects of policies such as population demographics and other socioeconomic considerations must be addressed.

RÉSUMÉ

Cette étude a été effectuée pour déterminer le potentiel d'assimilation des eaux usées par la rivière Nation-Sud. Une multitude de source ponctuelles d'eaux usées, tant municipales qu'industrielles, déversent déjà leurs effluents dans la rivière Nation-Sud. Les sources industrielles comptent présentement Ault Foods Ltée et Nestlé Ltée, tandis que la compagnie Fromages St-Albert Ltée viendra s'ajouter à cette liste. La plupart des municipalités qui déchargent présentement cherchent à accroître leur capacité, et d'autres municipalités planifient déverser elles aussi leurs eaux dans un avenir plus ou moins lointain. Toutes les municipalités traitent présentement leurs eaux au moyen de bassins de stabilisation conventionnels, communément appelés lagunes. Les municipalités qui comptent installer dans le futur des systèmes de collection et de traitement des eaux usées songent utiliser ce système elles aussi. La période de planification de cette étude est de 20 ans. Les industries ont fourni elles-mêmes leur projection d'accroissement. Les projections de populations utilisées proviennent de rapports précédents ainsi que de documents fournis par les diverses municipalités.

Le potentiel d'assimilation des eaux usées par une rivière est en quelque sorte la possibilité d'auto-purification inhérente à cette rivière. Ce potentiel peut être défini comme étant la capacité de transformer ou incorporer certaines substances (polluants etc.) par l'écosystème de sorte que la qualité de l'eau demeure à un niveau acceptable prédéterminé. On considère que la capacité d'assimilation est une ressource naturelle. L'exercice d'allouer ou diviser la capacité d'assimilation parmi plusieurs déchargeurs s'appelle l'allocation des décharges d'eaux usées.

Cette étude a examiné les performances passées des lagunes existantes. Certaines municipalités utilisent l'alun pour l'élimination du phosphore. La qualité du traitement des lagunes existantes est semblable à ce que l'on trouve en Ontario. Des données provenant de lagunes typiques ontariennes ont été utilisées pour arriver à un estimé conservateur de la qualité des effluents des lagunes. Les industries traitent leurs eaux au moyen de systèmes conçus pour leurs besoins spécifiques. Par le passé, ces systèmes ont eu des difficultés à se conformer aux normes imposées.

Toutes les données disponibles touchant la qualité de l'eau et aux débits de la rivière Nation-Sud ont été analysées. Suite à ces analyses, il a été déterminé que l'eau de la rivière Nation-Sud est basique, ayant un pH d'environ 7.5. La qualité de la rivière s'est améliorée depuis 12 ans. Cette amélioration est probablement due à la réduction de décharges d'eaux non-traitées dans la rivière, car plusieurs communautés se sont munies de systèmes d'épuration au cours de cette période. Par contre, les concentrations de phosphore sont constamment demeurées au-delà des normes gouvernementales. Les concentrations d'ammoniaque sont également restées élevées. Sur une base annuelle, 98% de l'ammoniaque et 93% du phosphore retrouvé dans la rivière provient de sources non-ponctuelles. Durant les périodes de déversement des eaux usées, par contre, plus de 50% de l'apport d'ammoniaque (la substance la plus problématique) provient de sources ponctuelles. De plus, lorsque le débit de la rivière

est très faible, la proportion d'ammoniaque provenant de sources non ponctuelles augmente sensiblement par rapport à l'apport total.

Les concentrations intrinsèques de certains polluants ont été estimées par des méthodes statistiques acceptées. La concentration intrinsèque de la demande biochimique d'oxygène (D.B.O.) est de 2.3 mg/L, tandis que celle de l'ammoniaque est de 0.18 mg/L.

Étant donné la concentration intrinsèque très élevée de phosphore, la rivière Nation-Sud n'a aucune capacité d'assimilation pour ce polluant. Toute nouvelle source ponctuelle d'eaux usées, qu'elle soit municipale ou industrielle, devra produire un effluent de très faible concentration en phosphore. De plus, les sources ponctuelles existantes devront améliorer leurs procédés de façon à augmenter autant que possible l'élimination du phosphore. Il est à noter que quelques lagunes ont commencé récemment à réduire leurs émissions de phosphore en ajoutant de l'alun à leurs procédés.

Le Ministère de l'environnement exige que les émissions totales de phosphore ne dépassent pas, pour un déchargeur donné, les limites imposées dans le Certificat d'autorisation du déchargeur en question. De plus, la concentration de phosphore de tout effluent ne doit pas dépasser 1 mg/L. Présentement, les communautés de Plantagenet, Winchester et la compagnie Ault Foods Ltée sont à la limite des restrictions imposées par leurs Certificats d'autorisation respectifs. On ne prévoit aucune augmentation d'émissions de phosphore. Une réduction des émissions actuelles est désirable.

Le potentiel d'assimilation des cours d'eau du bassin versant de la rivière Nation-Sud a été analysé sur une base des débits volumétriques 7Q20 et 30Q20. Une analyse statistique des débits de ces cours d'eau a été entreprise, et les débits de toutes les station de jaugeage ont été déterminés pour les saisons de l'automne et du printemps. En utilisant une fenêtre mobile de durée constante (soit 7 ou 30 jours), les hydrographes représentant les débits 7Q20 et 30Q20 ont été développés pour chaque station.

À partir de ces hydrographes, la période de dilution maximale de chaque station a été déterminée. Les débuts et les fins de ces périodes ne coïncidaient évidemment pas, étant donné la nature différente des cours d'eau à chaque station. Trois périodes ou fenêtres ont donc été définies. Les dates des débuts et fins de périodes furent utilisées pour définir ces fenêtres. La date de début la plus tardive et la date de fin la moins tardive bornent la fenêtre minimale (de durée minimale). La première des dates de début et la date de fin la plus tardive bornent la fenêtre maximale (de durée maximale). La moyenne des dates de début et la moyenne des dates de fin bornent la fenêtre moyenne (de durée moyenne). Une meilleure utilisation de l'hydrographe est possible en utilisant le concept des fenêtres multiples. Ceci résulte en une plus grande utilisation du potentiel d'assimilation de la rivière.

Pour chacune des stations, les débits volumétriques 7Q20 et 30Q20 de chacune des fenêtres mentionnées ci-haut ont été examinés. Les débits minimaux ont été retenus.

L'aire de chacun des bassins versants a été évaluée. Une corrélation a ensuite été dérivée entre les aires et les débits. Ces corrélations furent utilisées pour estimer les débits à tous les points d'intérêt. Les débits 7Q20 de l'automne ne furent pas calculés de cette façon, les données pour cette période étant trop éparpillées. Les débits 7Q20 pour l'automne furent calculés en multipliant le débit 7Q20 de l'automne à la station Plantagenet (estimé de façon statistique) par le ratio de l'aire du bassin versant au point d'intérêt à l'aire du bassin versant de la station Plantagenet.

En plus des bassins de stabilisation conventionnels, le système d'aération prolongée avec nitrification et le procédé New Hamburg furent considérés comme alternatives. Les effluents de ces trois types de traitement furent évalués afin de voir s'ils satisfaisaient les nouvelles normes de toxicité et de létalité contemplées par le Ministère de l'environnement. Ces normes s'appliquent sur l'effluent lui-même, avant même qu'il ne sorte de la conduite d'effluent. Afin de se conformer aux normes portant sur l'hydrogène sulfuré, toute lagune (existante ou projetée) devra se munir d'une cellule (petite lagune) d'aération. Il est aussi possible que les lagunes ne puissent produire un effluent conforme au normes portant sur l'ammoniaque, surtout lors de déversements printaniers. Malheureusement, le printemps est la période de déversement la plus importante, grâce à la fonte des neiges qui augmente les débits et du même coup la capacité d'assimilation. Les procédés d'aération prolongée et New Hamburg, quoique plus coûteux, produisent un effluent de meilleure qualité qui ne devrait avoir aucunes difficultés à rencontrer toutes les normes.

Les quantités d'eau de rivière disponible pour la dilution et l'assimilation de l'ammoniaque des eaux usées étaient insuffisantes chez certains déchargeurs. Le pH élevé de la rivière et la forte concentration intrinsèque d'ammoniaque contribuent à rendre l'ammoniaque le polluant limitant. Parmi ces déchargeurs, certains doivent relocaliser leur point de déversement à un endroit en aval où la dilution est plus grande. D'autres doivent améliorer la qualité de leur effluent en utilisant un traitement plus avancé (et plus coûteux).

Un modèle informatisé a été développé. La courbe en sac d'oxygène dissous fut trouvée grâce à une version modifiée de l'équation Streeter-Phelps. Cette équation incorporait, en plus des facteurs classiques, la demande azotée d'oxygène et la demande sédimentaire d'oxygène. La dégradation (ou assimilation) de la D.B.O. et l'ammoniaque fut caractérisée par des équations différentielles du premier ordre. La demande sédimentaire d'oxygène ne fut examinée que pendant l'automne, puisque les débits rapides et forts du printemps devraient en théorie débarrasser le lit riverain des matières biodégradables qui s'y sont déposées durant l'été. Le modèle tenait compte des débits d'eaux usées, des débits de base de la rivière et de ses affluents, des émergences d'eaux phréatiques etc.

En automne, les normes concernant l'oxygène dissous ne pouvaient être obéies même si tous les déchargeurs s'abstenaient de déverser leurs eaux usées. La faiblesse des débits automnaux, la concentration intrinsèque élevée de D.B.O. ainsi que la demande sédimentaire d'oxygène contribuent à cette situation déplorable. Toute possibilité de déversement automnal fut donc éliminée. La totalité des eaux usées devra donc être déversée pendant les fenêtres du printemps.

L'allocation des décharges d'eaux usées a été effectuée grâce au modèle pour les niveaux de population présents et futurs. La méthode consistait à maximiser l'utilisation de la capacité d'assimilation dans les région les plus en amont d'abord, puis de continuer successivement en aval.

Il fut déterminé que l'ammoniaque est le polluant limitant au printemps. Les résultats de l'optimisation sont les suivants:

- Pour les conditions présentes et futures, en utilisant les débits 7Q20 ou 30Q20, Williamsburg devra utiliser un système d'aération prolongée avec nitrification, car les faibles débits du drain McMartin ne peuvent assimiler l'effluent d'une lagune.

- Winchester devra relocaliser son point de décharge à la rivière Nation-Sud. Maxville et Ault Foods Ltée devront relocaliser leurs points de décharge plus en aval sur la rivière Castor.

- Pour les conditions présentes et projetées, en utilisant les débits 30Q20, toutes les autres communautés pourront faire le traitement de leurs eaux usées au moyen de lagunes conventionnelles.

- Pour les conditions présentes, en utilisant les débits 7Q20, toutes les autres communautés pourront faire le traitement de leurs eaux usées au moyen de lagunes conventionnelles.

- Pour les conditions futures, en utilisant les débits 7Q20, les communautés d'Embrun et de Moose Creek (en plus de Williamsburg) devront utiliser des systèmes d'aération prolongée avec nitrification. Toutes les autres communautés pourront faire le traitement de leurs eaux usées au moyen de lagunes conventionnelles.

- Un scénario de déchargement par étapes a été développé. Plus les conditions sont sévères, (en commençant par la condition présente avec les débits 30Q20 et en finissant avec la condition future avec les débits 7Q20) plus l'utilisation par étapes des trois fenêtres de base (maximale, moyenne et minimale) est nécessaire. L'utilisation par étapes des fenêtres permet une meilleure utilisation de l'hydrographe du printemps, donc un meilleure usage de la capacité d'assimilation de la rivière.

Une analyse de la zone de diffusion fut également entreprise. Il fallait voir si, lors de la diffusion des eaux usées traitées dans la rivière réceptrice, la concentration de l'ammoniaque ne violait aucune norme sur au moins 75 pour-cent de la surface de toute section transversale le long de la rivière. Cette analyse se bornait à l'ammoniaque, qui est le polluant limitant. L'objectif d'une zone de passage de 75 pour-cent peut être atteint partout le long de la rivière Nation-Sud pour les conditions présentes et futures aux débits 30Q20, et pour les conditions présentes aux débits 7Q20. Aux débits 7Q20, pour les conditions futures, les tronçons les plus en amont

ne peuvent pas fournir une zone de passage de 75 pour-cent. La plupart des affluents ne pouvaient fournir cette zone de passage sous aucunes conditions.

Une évaluation comparative des coûts des diverses options de traitement a été entreprise. Les traitements par aération prolongée avec nitrification ainsi que le procédé New Hamburg devraient avoir des coûts capitaux très semblables.

Les recommandations majeures découlant de cette étude sont les suivantes: un programme intensif de surveillance de la qualité des eaux des rivières du système Nation-Sud devrait être mis sur pieds; une analyse stochastique ayant pour but de définir la fréquence et durée des conditions nuisibles à la santé de la rivière devrait être entreprise; une étude approfondie des sources non ponctuelles de pollution devrait être entreprise afin d'éventuellement aboutir à la mise en oeuvre de mesures visant à diminuer l'apport non-ponctuel de pollution.

La possibilité de contrôler et faire varier les débits de déversement d'eaux usées au prorata des débits de la rivière offre le potentiel d'augmenter sensiblement le degré d'utilisation de la capacité d'assimilation des cours d'eaux.

Il est impératif d'établir des politiques claires sur les questions d'allocation des décharges d'eaux usées, ainsi que l'allocation des ressources hydriques. En plus des effets que ces politiques peuvent avoir sur la quantité des décharges permises, leurs traitements et coûts résultants, il faut considérer les enjeux socio-démographiques, politiques et socio-économiques de ces politiques.

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FIGURE 7-1 Typical Waste Plume

SECTION 1

INTRODUCTION

INTRODUCTION

1.1 BACKGROUND

In June 1991, Gore & Storrie Limited (G&S) prepared a detailed proposal for the South Nation River Conservation Authority's (SNRCA) Wastewater Allocation Study for the South Nation River watershed. The SNRCA subsequently authorized G&S to proceed with the study based on the Terms of Reference (TOR) provided by the SNRCA and on the methodology contained in the proposal.

1.2 TERMS OF REFERENCE

TOR for the study are included in Appendix A. The overall objective of the study was to establish a watershed management strategy for wastewater discharges to maintain stream quality standards. Present and future wastewater production conditions were to be considered. Low flow 7Q20 and 30Q20 streamflow conditions were specified for stream dilution assessment. Constraints imposed on the study included the use of constant discharge rates for the point sources and Certificate of Approval (C of A) effluent limits.

1.2.1 Point Sources

The watershed and locations of municipal and industrial treatment facilities within the study area are shown in Fig. 1-1. As well, the locations of proposed treatment facilities and the major proposed expansions to existing plants are indicated.

The existing community point source discharges along with the stream into which they discharge are as follows:

- **Casselman** (South Nation River)
- **Chesterville** (South Nation River)
- **Embrun** (Castor River)
- **Maxville** (West Branch Scotch River)
- **Plantagenet** (South Nation River)
- **Russell** (Castor River)
- **Spencerville** (South Nation River)
- **St. Isidore** (East Branch Scotch River)
- **Williamsburg** (McMartin Drain)
- **Winchester** (Henderson Drain)

The existing industrial discharges to the South Nation River are:

- **Ault (Winchester)** (Anabel Drain)
- **Nestles (Chesterville)** (South Nation River)

Future treatment plants are proposed for the following municipalities:

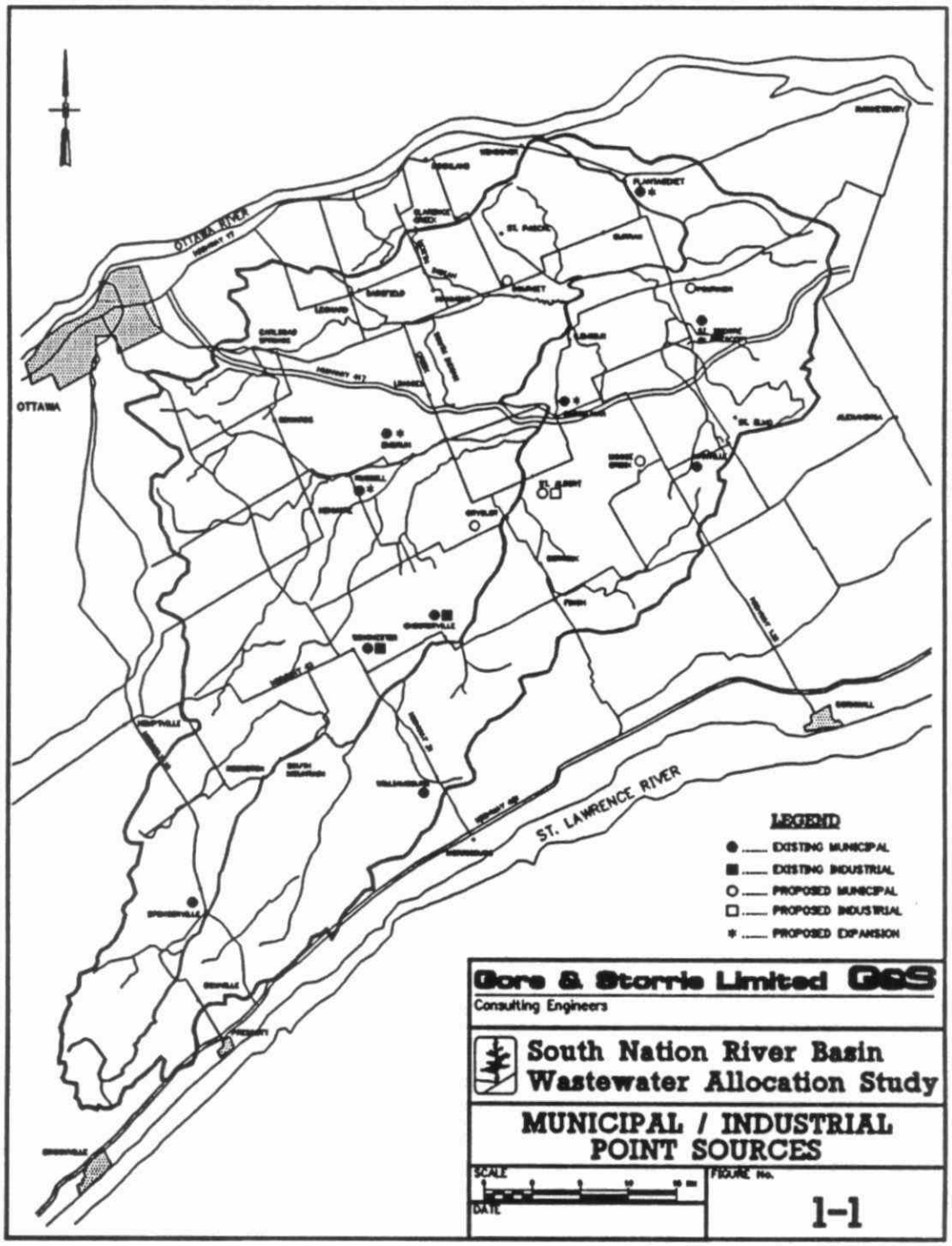

Figure 1-1 Present and future point sources in the South Nation watershed

- **Bourget** (Bear Brook)
- **Crysler** (South Nation River)
- **Fournier** (South Nation River)
- **Moose Creek** (Moose Creek)
- **St. Albert** (South Nation River)

The following industrial treatment plant is proposed for the future:

- **St. Albert Cheese (St. Albert)** (South Nation River)

1.2.2 Wastewater Treatment

Lagoons are currently used to treat wastewater for all municipalities. Industries are using more sophisticated, custom designed wastewater treatment processes. The lagoons are confined to discharge periods of the spring and fall when streamflows normally are high. The study was to examine staging discharges and consider alternative wastewater treatment processes or relocation of discharge points as required to meet the stream quality objectives under low flow conditions assessed by 7Q20 and 30Q20 analyses.

SECTION 2

WASTEWATER TREATMENT IN THE SOUTH NATION WATERSHED

WASTEWATER TREATMENT IN THE SOUTH NATION WATERSHED

The characteristics and performance of the existing wastewater facilities were examined to provide background information for the study. Influent and effluent water quality data and flows for each operating facility were obtained from Ministry of the Environment (MOE) records.

Site visits were made to most facilities and discussions were held with the operators to obtain information on operational procedures used and other pertinent comments on the operation of the facilities. Additional data on wastewater quality were also obtained from the operators. A list of each of the facilities which was visited on the tour is included in Appendix B, along with the operator who runs the facility.

2.1 CERTIFICATION REQUIREMENTS

The fundamental document specifying effluent quality limits, operating conditions and other special conditions is the Certificate of Approval (C of A) issued by the MOE. Copies of the Cs of A for all communities and industries that currently have wastewaer treatment facilities are given in Appendix C.

The four primary sources of data included MOE reports (MOE, 1986-89; MOE, 1990a), Cs of A and the MOE Laboratory Data Sheets which were obtained directly from the plant operators and operating authority staff. Other basic data on the type of facility at each location as well as the connected population were obtained from these sources for each treatment facility. Conventional facultative lagoons with 6 or 12 months of storage capacity are the wastewater treatment processes for all of the municipalities. The industries have more sophisticated wastewater treatment processes but they also have storage lagoons at the end of their process train to retain effluent during periods of low flow in receiving waters. Table 2-1 provides general information on the existing facilities. Appendix D contains a summary table for each facility.

The primary pollutants of concern in this study are total phosphorus (TP), biochemical oxygen demand (BOD), suspended solids (SS), hydrogen sulphide and ammonia. C of A effluent quality criteria for each facility are given in Table 2-2. The rated capacity of the treatment facilities along with other quantity characteristics and mode of phosphorus precipitation are given in Table 2-3. MOE (personal communication, R. Watson, 1992) reports that design ratings for lagoons are generally based on filling the lagoon to the emergency overflow level. A common standard should be developed for rating the capacity of lagoons and the effective rated capacities of existing lagoons should be reviewed.

It was noted that values for effluent criteria and design capacities differed between the MOE discharge reports (Green Book; MOE 1986-89) and the MOE status report (MOE, 1990a). It was also noted that the effluent discharge criteria presented in the MOE Sewage Compliance Status Report (MOE, 1990a) did not necessarily agree with

the criteria indicated on the Cs of A. As well, in some cases, the design capacities (lagoon volume and discharge rating) in the two documents differed. Table 2-4 summarizes the information from the two sources.

Table 2-1 General Facility Characteristics

Facility	Discharge To	Population Served	Facility Type	Discharge	Operator
MUNICIPAL					
Casselman	South Nation River	2,400 (1991)	Two Cell Lagoon	Seasonal	MOE
Chesterville	South Nation River	1,500 (1991)	Two Cell Lagoon	Seasonal	MOE
Embrun	Castor River	3,500 (1989)	Six Cell Lagoon	Annual	Municipal
Maxville	West Branch Scotch River	800 (1991)	Two Cell Lagoon	Annual	MOE
Plantagenet	South Nation River	900 (1991)	One Cell Lagoon	Seasonal	MOE
Russell	Castor River	1,800 (1991)	Two Cell Lagoon	Seasonal	MOE
Spencerville	South Nation River	350 (1991)	Two Cell Lagoon	Annual	MOE
St. Isidore	East Branch Scotch River	700 (1991)	Three Cell Lagoon	Annual	MOE
Williamsburg	McMartin Drain to South Nation River	300 (1989)	Two Cell Lagoon	Annual	Municipal
Winchester	Henderson Drain to East Castor	2,200 (1991)	Five Cell Lagoon	Seasonal	MOE
INDUSTRIAL					
Ault (Winchester)	Annable Drain to East Castor River	NA[1]	Aerobic Biological Treatment	Annual	Ault
Nestle (Chesterville)	South Nation River	NA	Secondary Treatment Aerated Lagoon	Continuous except June 15 to Oct 15	Nestle

1. NA - not applicable

Table 2-2 Effluent Quality Specified in Certificates of Approval

Facility	BOD_5 mg/L	SS mg/L	TP[1] mg/L	H_2S mg/L	NH_3-N mg/L
MUNICIPAL					
Casselman	30.0	40.0			
Chesterville[2]					
Embrun	30.0				
	(1,829 kg/d)				
	(19,700 kg/yr)				
Maxville	30.0	30.0	1.0		
	(4,932 kg/yr)	(4,932 kg/yr)	(164 kg/yr)		
Plantagenet					
Russell					
Spencerville	25.0	30.0	1.0	5.0	20.0
St. Isidore					
Williamsburg	30.0	30.0			
Winchester	30.0	30.0	1.0	0.0	20.0 - 2.3[3]
	(9,720 kg/yr)	(9,720 kg/yr)	(469 kg/yr)[4] (648 kg/yr)[4]	0.0	(6,480 - 745 kg/yr)
INDUSTRIAL					
Ault (Winchester)	5.0	15.0	1.0	0	5.0
	(807 kg/yr)	(2,425 kg/yr)	(162 kg/yr)	(0 kg/yr)	(807 kg/yr)
Nestle (Chesterville)	15.0	15.0	1.0		

1. MOE policy currently stipulates TP effluent at 1.0 mg/L.
2. MOE Policy 08-01 currently stipulates 30 mg/L BOD_5, 40 mg/L SS.
3. Spring - fall discharge.
4. For an annual discharge (once per year), the C of A allows a load of 469 kg to be discharged. For semi-annual discharges (twice per year) each discharge may contain 324 kg.

Table 2-3 Discharge and volume characteristics of treatment facilities

Facility	Rated Capacity[1] (m^3/d)	Volume (m^3)	Area (ha)	Phosphorus Precipitation	Allowed Discharge Time Span
MUNICIPAL					
Casselman	1,346	243,000[2]	16.2	Batch	2 times per year
Chesterville	1,040	185,040	11.9	Batch	2 times per year Spring on/before May 7 Fall after Nov. 1 Dilution rate 13:1
Embrun	1,798	800,000[2]	53.4	None	Spring between April 1-21
Maxville	450	164,025	9.8	Batch	Spring 21 days - max 95 L/s 30 days - max 63 L/s Dilution rate 3:1
Plantagenet	590	100,000[2]	6.9	Batch	2 times per year
Russell	1,000	237,000[4]	15.8	Batch	Seasonal
Spencerville	318	116,000	6.9	Continuous	Spring (8-30 days) March 15 - April 21
St. Isidore	655	220,000[2]	14.6	Batch	Annual
Williamsburg	205	75,000[2]	4.9	NIL	Spring not before March 15 and not after April 21
Winchester	1,725	470,000	19.0	Continuous	March 15 - Apr. 30 Nov. 1 - Nov. 30 Flows > 0.03 m^3/s
INDUSTRIAL					
Ault (Winchester)	573	275,632[3]	10.1	Continuous	Spring March 15 - April 15
Nestle (Chesterville)	2,045 (max)			Continuous	October 15 - June 15

1. Certificate of Approval rating.
2. Obtained from plant operators.
3. Lagoon volume only. Does not include volume of other treatment units.
4. Personal communication MOE (R. Watson, 1992).

Table 2-4 Effluent Criteria and Discharge Ratings

Facility	**MOE Compliance** Effluent Limits BOD/SS/TP (mg/L) Flow (m^3/d)	**C of A** Effluent Limits BOD/SS/TP (mg/L) Flow (m^3/d)
Casselman	30/40/1.0 1,364	
Chesterville	30/40/1.0 1,030	
Embrun	30/40/NA 1,600	30/NA*/NA
Maxville	30/30/1.0 1,640	30/30/1.0 450
Plantagenet	30/40/1.0 590	
Russell	30/40/1.0 1,000	1,000
Spencerville	NA NA	25/30/1.0
St Isidore	30/40/1.0 655	
Williamsburg	30/40/NA 290	30/30/NA 290
Winchester	30/40/1.0 2,611	30/30/1.0 1,725
Ault	5/15/1.0 NA	5/15/1.0 573
Nestles	15/15/1.0 2,045	15/15/1.0 2,045

* NA - not available

2.2 SITE INFLUENT AND EFFLUENT CHARACTERISTICS

Influent and effluent quality and flow data from the reports noted above were reviewed and summarized. Nitrogen effluent data were generally unavailable from the above sources. A supplementary data set on nitrate, nitrite, total ammonia (NH_{3T}) and total Kjeldahl nitrogen (TKN) in the lagoon effluents was obtained from MOE (MOE, 1991a).

During the course of the data collection and assimilation component of this study, several discrepancies were discovered in the data reported from the various sources. During the site visits, it became apparent that some of the plant operators thought that summary data (taken from MOE, 1986-89) presented to them were not necessarily accurate. Following this, the operators provided the MOE Laboratory sheets so that validation of the data could be undertaken. Data from the following plants were provided:

- Casselman
- Embrun
- Plantagenet
- Williamsburg

Influent flow and loading information for 1990 are given in Table 2-5. Effluent quality for the facilities during 1990 is reported in Table 2-6. More detailed data including data from other years of record are available in Appendix D. Graphs of effluent quality are also shown in this appendix.

In general, the data between the two sources agreed very well. There appeared to be a small number of "rounding" errors which are not considered to be of significance. At Winchester, flow records for 1986-89 were not used because bypass flows were not monitored and included in the measured flows. Therefore flows during wet weather periods in particular, are not accurate. After 1990 flow metering measured the total flow for the community.

Major discrepancies were found for the Embrun facility. The MOE discharge summary (MOE, 1986-89) for 1987 indicated that there was no discharge in 1987. However, the MOE Laboratory records indicate that a discharge did in fact occur during this year. Additionally, the data for the discharge period of March and April 1988 between the two documents differed as follows:

	Concentration (mg/L)	
	MOE Green Book	MOE Laboratory Sheet
BOD Ave Effluent	9.50	13.80
SS Ave Effluent	8.70	16.70
TP Ave Effluent	1.73	1.73

Table 2-5 Influent flows and loading rates - 1990

Facility	Ave Daily Flows (m^3/d)	Ave Per Capita Flow ($m^3/cap/d$)	Calculated Domestic Load (kg BOD/d)	Calculated Loading Rate (kg BOD/ha/d)
MUNICIPAL				
Casselman	1,000	0.42	176	11.0
Chesterville	811	0.54	120	9.9
Embrun	1,300[1]	0.32	320	6.0
Maxville	164	0.21	>64	4.0
Plantagenet	617	0.69	72	10.4
Russell	529	0.29	160	10.1
Spencerville	150[1]	0.40	29	4.2
St. Isidore	348	0.50	>64	4.6
Williamsburg	32[1]	1.06	24	4.9
Winchester	1,788	0.81	176	10.3
INDUSTRIAL				
Ault (Winchester)	900	NA[2]	NA	NA
Nestle (Chesterville)	830	NA	NA	NA

1. Obtained from plant operators.
2. NA - not applicable.

Table 2-6 Effluent quality - 1990

Facility		BOD_5 mg/L	SS mg/L	TP mg/L	Q m^3/d	H_2S	NH_3-N[1]
MUNICIPAL							
Casselman	AVE	7.13	11.90	0.27	6,098 (60 days)		
	MAX	9.25	16.30	0.30			
Chesterville	AVE	8.85	23.15	0.43	4,513 (60 days)		
	MAX	10.00	37.30	0.52			
Embrun[2]	AVE						
Maxville[2]	AVE						
Plantagenet	AVE	11.23	27.33	0.41	1,877 (120 days)		
	MAX	17.70	38.83	0.56			
Russell	AVE	3.30	25.30	0.35	6,427 (30 days)		
	MAX	3.30	25.30	0.35			
Spencerville[2]	AVE						
St. Isidore	AVE	6.00	21.50	0.63	2,120 (30 days)		
	MAX	7.00	24.00	0.71			
Williamsburg[2]	AVE						
Winchester	AVE	7.65	10.65	0.31	10,875 (60 days)		
	MAX	8.30	11.30	0.34			
INDUSTRIAL							
Ault (Winchester)	MAR/APR (1991)	9.8	28	0.94	9,970 (20 days)		2.9
Nestle (Chesterville)	AVE (1989)	22.7	34.7	0.8	623		2.2
	MAX (1989)	33.0	80.0	1.3	1,025 (270 days)		8.2

1. NH_3-N data for 1990 were only available for Nestle.
2. Data are unavailable for 1990.

There were also some errors which related to the displacement of a decimal point in the data presented in MOE (1986-89) and the MOE Laboratory documents. These have been corrected in the data presentations.

Alum treatment for phosphorus removal was generally initiated during 1989 at most of the facilities. It was noted by more than one operator that effluent quality does deteriorate as the last 20-30 cm of water is withdrawn from the ponds during discharge. The frequency of sampling influent and effluent for the lagoon systems for all parameters increased from 1989 to the present.

2.3 DESCRIPTION OF COMMUNITIES AND WASTEWATER COLLECTION AND TREATMENT FACILITIES

The following sections contain information briefs on each of the communities and industries and their wastewater treatment facilities. The majority of information was obtained from the site investigations and discussions with the operators.

Facultative stabilization ponds without any mechanical aeration are typically designed for organic loading rates of 20-70 kg/ha/d based on a detention time of 5-30 d (Metcalf & Eddy, 1991). During cold weather conditions there is little treatment. However, all of the ponds in the watershed are designed for a minimum of six months retention and these loading rates would be conservative on that basis.

Hydraulic loading rates on an areal basis (kg/ha/d) are also often specified as a design criteria. Given the constraint that discharges are confined to the spring and fall, hydraulic loading rates are limited to rates that produce a minimum of six months retention in the ponds. Also retention time is the more relevant parameter dictating treatment. Retention times of six months are well beyond a detention time of 30 days.

2.3.1 *Casselman*

The community of Casselman is located near the centre of the drainage basin, where the major east-west road (Hwy 417) crosses the South Nation River. It is a community of about 2,400 people. There is potential for growth and the community has been growing rapidly; the community is within commuting distance to the large urban centre of Ottawa.

Sanitary domestic wastewater from this community is collected in a sewer system, and is all transferred to one pumping station. From there, it is discharged to a lagoon system consisting of two rectangular cells, each with a surface area of 8.1 ha for a total of 16.2 ha.

Discharge from the lagoons is allowed seasonally (i.e., spring and fall). The contents of lagoons(s) to be discharged are batch treated with alum, allowed to settle, and then discharged at a controlled rate to the South Nation River.

The present average daily flow is approximately 1.0 ML/d. Values for 1991 may be slightly lower, but this is likely due to the unusually dry conditions during the 1991 summer. A flow of 1.0 ML/d is suggested for the current flow. During wet weather, wastewater flow increases to 1.3-1.5 ML/d.

Batch treatment with alum has provided effective enhancement of the lagoon effluent quality. Alum dosage during batch treatments in 1990 was approximately 90 mg/L which is normal

for this physical-chemical treatment method. Records indicate that overall effluent characteristics of 10/15/0.5 (BOD/SS/TP) may be expected from this plant.

The storage capability provided in the current two lagoons is 243 ML assuming a depth of 1.5 m. At an average daily flow of 1.0 ML/d, storage capacity is 243 days. This is adequate for the seasonal discharge which is allowed from this facility.

Sludge accumulation has reached an average depth of about 15 cm. This layer will be removed during the next 2 years at a rate of one cell per year.

2.3.2 *Chesterville*

The Chesterville water pollution control plant (WPCP) receives the sanitary domestic wastewater of this community of about 1500 people. Nestle is located in this community but its industrial process wastewater is treated in a completely separate wastewater treatment plant.

The Chesterville WPCP is a two cell facultative lagoon system. Each cell has a surface area of just under 6 ha. Discharge from this lagoon system is allowed two times per year, in the spring and fall. The lagoons are operated in series and effluent is discharged to the South Nation River.

The treatment facility has served this community for a considerable period of time. The first cell was constructed about 1970; the second cell was added in the early 1980s. One cell has been cleaned once. Sludge accumulation was scraped from the bottom at the inlet end, and was pushed up onto the berm of the lagoon. No sludge has been taken off-site.

The facilities are operated by MOE staff; headquarters for this operation staff group are in Chesterville itself.

The treated and stored wastewater in the lagoons is batch treated with alum, allowed to settle, and then discharged to the receiving stream in spring and fall. Alum dosage for batch treatment has been about 80-100 mg/L, which is in the normal application range for lagoons at this loading.

Weed growth is becoming difficult to control at this site. Weed growth produces islands of weeds in stabilization ponds and decreases the volumetric capacity of the ponds. Channelization of flows can occur and wind action supplying aeration is diminished. Uncontrolled weed growth can eventually render the pond into a swamp: odours, insect breeding and reduced treatment will result.

The exceptionally dry summer of 1991 has made it difficult (in some lagoons almost impossible) to sustain adequate water depth for good weed control. Some intensive work will be needed at several sites, including Chesterville, to remove weed growth from the lagoon bottoms.

The organic loading to this set of lagoons has been higher than loadings at other (annual discharge) facilities. However, its loading of approximately 10 kg BOD_5/ha/d is still

relatively low in absolute terms, and excellent treatment results have been achieved. Batch treatment is now routinely used to control SS and TP discharges and good effluent quality of roughly 15/30/0.5 is routinely achieved.

The storage volume in both cells is roughly 185 ML, assuming an average depth of 1.5 m in the lagoons. At an average flow of 0.8 ML/d, there is a total storage time of 230 days between discharges. The facility is conservatively loaded in terms of organic loading, and has limited spare capacity in terms of hydraulic capability.

The C of A for the Chesterville plant states that the spring discharge is to coincide with the peak river flows and be completed on or before the 7th day of May of each year. The spring discharge rate is to equal the maximum hydraulic capacity of the outfall. The fall discharge is not to commence prior to the first day of November and no discharge is to occur when flow in the receiver at Chesterville is less than 0.2 m^3/s. At river flow rates between 0.2 and 1.84 m^3/s the discharge rate is to be regulated in accordance with river flow. At river flow rates above 1.84 m^3/s, the discharge rate is to equal the maximum hydraulic capacity of the outfall.

2.3.3 Embrun

Embrun is the largest community (in terms of population) within the South Nation drainage basin. It is located near the centre of the geographical area, and has about 4,000 inhabitants.

Domestic wastewater from this community is collected in a sanitary sewer system, and is pumped to a six cell lagoon system. The complete facility was constructed in 1985 and has been in service since that time. Effluent is discharged on an annual basis during the spring to the Castor River. No metal salts are added (continuously or on a batch basis) to the wastewater for phosphorus and sediment removal at this time.

The whole collection system, pumping system, and lagoon system was constructed during the 1980s. The flow from the 4,000 serviced population was estimated to be 1.3 ML/d. The per capita flow is 320 L/cap/d, which is indicative of a new, tight sewer system. The lift stations and pumping station are also relatively new and built to the standards and requirements of their construction years.

The six lagoon cells are designed as four primary cells and two secondary cells. Gross surface area is 53.4 ha (the C of A indicates that the lagoon surface area is 37.2 ha). The current organic loading is 6.0 kg BOD_5/ha/d which is conservative. Good quality effluent may be expected.

Recorded results of effluent quality could not be extracted from MOE data but they were provided by municipal supervisory staff to the G&S office in Ottawa. The system has been functioning within the 20/30 effluent range for BOD and SS. TP has usually been less than 1 mg/L according to the lab data sheets.

The total storage capacity in the six cells is roughly 800 ML at an assumed depth of 1.5 m. At current average flow of 1.3 ML/d, that implies storage for 530 days. The hydraulic loading can be allowed to increase to about 1.89 ML/d before alterations to discharge rules and/or physical facilities will be required.

The facilities are owned and operated by the municipality. The overall approach appears to be one of active operation and maintenance. Improvements and modifications to the facilities have been found necessary and then quickly implemented.

Wind and/or ice action has caused damage to some areas of the lagoon embankment. Municipal staff have authorized and carried out upgrading by installing rip-rap in some of the most affected areas, and also by covering some embankments with geo-textile. It is expected that geotextile reinforcements will become more popular to strengthen banks and for weed control.

The current facility is relatively "young", conservatively sized, and capable of serving a larger population while maintaining effluent quality at current concentration levels.

2.3.4 Maxville

The community of Maxville is situated on the eastern limit of the South Nation drainage basin. Approximately 800 connected population members are serviced by this recently constructed (1989) WPCP.

The total facility is made up of a sewer (collection) system, two pumping stations, and a two-cell facultative lagoon system. Discharge from this system is also on an annual basis during spring. The treated and stored wastewater is batch treated by alum addition, and then allowed to settle before discharge to the west branch of the Scotch River.

The treatment facility is relatively new. It has been in service only three years with little time for significant sludge and grit accumulation. The facility is operated and maintained by MOE staff members, who are based at the Casselman WPCP.

Some of the wastewater from the slaughterhouse operation (mentioned in the comments for the St. Isidore WPCP below) is also brought to the Maxville facility. The industrial wastewater is discharged directly into the lagoons, and therefore the analytical results of the "raw sewage" do not include the industrial wastewater component. Addition of this (potentially) high strength wastewater may have a significant influence on lagoon loading, effluent quality and sludge accumulation. However, the slaughterhouse operation was terminated in 1991.

The two rectangular cells have a total surface area of 9.75 ha. There is little room left on the present plant site for any future expansion. The storage volume in the two existing lagoons is roughly 164 ML. At the current average daily flow of 0.16 ML/d, the storage volume is sufficient for 1025 days. The existing facilities are able to handle a hydraulic loading up to 0.45 ML/d while still providing more than sufficient storage.

The organic loading (kg BOD_5/ha/d) is very low at this site due to its recent design and construction. Therefore high quality effluent is expected from this WPCP. Effluent quality capability is at least 20/20/0.6 at current loadings.

2.3.5 *Plantagenet*

This community is situated in the northern tip of the South Nation River drainage basin, and fairly close to the discharge point of the South Nation into the Ottawa River. It is a relatively small community with a present connected population of approximately 900 people.

There is one small sewage lift station in the community, along with one pumping station which transfers the wastewater to a single 6.9 ha lagoon. Batch treated lagoon effluent can be discharged 2 times per year in the spring and fall. The discharge is routed directly to the South Nation River.

The average daily wastewater flow is 0.65 ML/d (or just above 700 L/cap/d). This flow rate is high. It indicates that there is likely a major input of infiltration/inflow (I/I) into the sewer system even when it is not raining (or melting). This same tendency is indicated by much higher wet weather flows which are double to triple the average daily flow.

The organic loading to the lagoon system is an acceptable 10.4 kg BOD_5/ha/d. The storage capacity of the lagoon is 100 ML. It allows storage for 153 days at average flow which is not sufficient for seasonal discharge.

Batch treatment with alum is now routinely used to enhance effluent quality prior to discharge. Results have been adequate at approximately 15/25/0.5. While this facility is within its C of A compliance limits, effluent quality is not as good as that obtained at other plants in the drainage basin. This can be attributed to:

(a) lack of storage (and treatment) time,
(b) single cell operation, which makes it necessary to "feed" the lagoon during discharge.

It has become evident that major areas of concern include:

(a) apparent I/I problems in the collecting system,
(b) the need for at least one more treatment/storage cell.

2.3.6 *Russell*

This community is on the west side of the drainage basin, and is the closest community to the city of Ottawa which enhances its growth potential as a bedroom community to Ottawa. The growth rate in this community has been relatively high. There is a political desire for growth and expansion and because of its proximity to Ottawa there is demand for housing units. The net effect has been rapid growth. The present serviced population is about 2,000 people.

The WPCP is provincially owned and operated by MOE staff. The treatment facility consists of a collection system, a pumping station, and a two cell lagoon system. Annual discharge to the Castor River is allowed. Each discharge is preceded by batch treatment with liquid alum. The average daily wastewater flow is now approximately 0.6 ML/d or 300 L/cap/d which is a typical value for a residential area.

The operator indicated that there is a significant increase in the wastewater flowrate during wet weather conditions. I/I is deleterious to good treatment because

(a) I/I water demands sewer and pump station capacity,
(b) I/I water consumes lagoon storage capacity especially for spring discharges.

If a detailed evaluation of wastewater flow records shows that very high I/I is in fact occurring, it will probably be beneficial to conduct specialized and intensive work to reduce this influent flow.

Batch treatment with alum to precipitate phosphorus and to enhance solids removal was started in this system during 1989. Records indicate that the expected effluent quality was achieved. Effluent quality of 10/20/0.4 may be expected from this lagoon facility.

The WPCP was built around 1979. This 10 year old facility is still well within the usual lifespan of lagoon WPCP units.

The storage volume available in the two cells is 235 ML, based on an assumed average depth of 1.5 m in the lagoons. At an average daily flow of 0.6 ML/d, the current storage capacity is adequate for 390 days. This implies that the existing facility is still conservatively loaded on an organic loading basis, but that the units are virtually fully loaded hydraulically. If population growth continues, it will soon be necessary to implement seasonal (2 times per year) discharge, or to provide more storage capacity to enable continued annual discharge.

2.3.7 Spencerville

Spencerville is at the southwestern tip of the drainage basin, close to the head water of the South Nation River. The serviced population is estimated to be 350 people.

Sanitary wastewater is collected at a pumping station and is pumped to a two cell lagoon system at the southeast corner of the municipality. Construction of the facility has barely been completed, and no historical data are available at this time. The two cells have a combined surface area of 6.9 ha. The storage volume in the lagoons is about 116 ML.

Flow measurement records during the summer of 1991 have shown that the current daily flow is roughly 0.15 ML/d which is a typical flow of just over 400 L/cap/d. Experience in coming months will have to confirm these flows, and also provide information on wet weather conditions. At this time, estimating the annual average daily flow at 0.18 ML/d is reasonable. The total volume available is estimated at 116 ML. Based on that value the lagoon storage capacity is adequate for 640 days. This is a conservative value for the annual discharge condition for which this plant has been designed.

Alum is used at this plant to provide phosphorus precipitation and to enhance solids removal. It is continuously dosed into the lagoon influent. Operating experience is required to fine-tune the applied chemical dosage; it is adjustable over quite a large range.

It is estimated that the facilities as they exist today are capable of routinely producing 15/15/0.6 effluent. In summary, this facility was recently constructed, conservatively sized, and may be expected to produce effluent well within the C of A limits.

2.3.8 *St. Isidore*

The three cell lagoon system in this community receives domestic sanitary wastewater from roughly 800 inhabitants, plus intermittent input of slaughterhouse wastewater which is trucked to the site and discharged directly into one of the lagoons (#1 or east lagoon during the site visit). The slaughterhouse operation was terminated in 1991.

One sewage pump station, located within the community, pumps domestic wastewater to the lagoon site. The station contains two pumps: one service and one standby. Standby power is provided. The pumping station is adequate for current needs.

The facility is operated by MOE staff stationed at the Casselman WPCP. The lagoon system is now roughly ten years old. It has performed well throughout this time span. No sludge removal has been found necessary during that time frame.

Discharge from this lagoon system is allowed annually (in spring). The contents of the lagoon(s) to be discharged are batch treated with alum (and allowed to settle) prior to discharge. The apparent alum demand for batch treatment was very high; the dosage during 1990 was above 200 mg/L. At least two factors contributed to this high dosage:

(a) relatively high organic loading due to slaughterhouse waste, leading to significant algae growth.

(b) high P input from the same source.

The facilities are well kept and maintained. It is evident in this facility also that it is becoming progressively more difficult to control weed growth in lagoon systems. There is copious weed growth on the lagoon banks and the beginning of weed growth on the lagoon bottoms. (See comments on weed growth in Sec. 2.3.2).

Fairly high quality effluent is expected from this lagoon facility and is achieved with batch alum treatment. Records for 1990 indicate effluent quality of 10/20/0.6.

Using an assumed depth of 1.5 m, the storage volume in the three lagoons is roughly 220 ML. Average daily flow of 0.3 ML/d implies that the presently available storage capacity is adequate for about 730 days. This is sufficient for annual discharge at the present flows.

2.3.9 *Williamsburg*

The rural community of Williamsburg is very close to the southern boundary of the watershed. The municipal core is roughly 8 km north of Highway 401. Williamsburg is a well established community of about 300 people.

A municipal WPCP facility was constructed in 1988. It consists of a pumping station, a forcemain, and a two cell lagoon system owned and operated by the municipality. Wastewater is treated in the lagoons, and is also stored there for annual discharge. Effluent requirements are focused on removal of BOD_5 and SS; there is no requirement for P removal at this facility at this time.

Wastewater (lagoon influent) flow is not measured; the only method of quantifying the hydraulic loading is based on wastewater pump running time. Based on this value, it is estimated that the daily wastewater flow is approximately 0.32 ML/d which is equivalent to a per capita flow of 1060 L/cap/d. It is questionable that such a high per capita flow would actually occur, especially during the dry summer conditions experienced in 1991. It is suggested that confirmatory work be planned to more accurately establish the actual wastewater flow.

Organic loading to the lagoon system is low at 4.9 kg BOD_5/ha/d. Good treatment results may be expected under such conditions and have in fact been noted.

The two lagoons were constructed in 1988. They were dug in the native soil at the northwest end of the built-up community. Clay for lining the lagoons was extracted from a neighbouring property and was used to construct basins which would retain the wastewater for its annual discharge to the receiver.

From the operator's comments, the water level in the two lagoons has virtually not increased since the end of April, 1991. This was attributed to evaporative loss. There is no doubt that the warm, dry summer has enhanced loss by evaporation but it is doubtful whether it is reasonable to attain "zero gain" by evaporation only. It is suggested that testing is needed to determine whether lagoon leakage may have played a role.

The surface area of the two lagoons is roughly 4.9 ha. The available storage volume can therefore be estimated at 75 ML, using an assumed depth of 1.5 m. If the average daily flow is assumed to be 0.12 ML/d (based on 400 L/cap/d for 300 people), it is estimated that the existing lagoons can provide storage for 600 days. These estimates show that the lagoon facility serving this community is conservatively sized if the hydraulic loading can be maintained at reasonable per capita quantities.

Routine sampling and sample submission of influent and effluent (during discharge) is carried out at this plant. However, during the site visits the data were not available. It is estimated that the lagoon system as it is today can routinely produce 20/20/3 effluent based on the results obtained at other similar facilities.

Chemical enhancement with continuous or batch alum treatment could be used to attain 10/10/0.5 effluent.

2.3.10 Winchester

Winchester is a rural community of approximately 2,200 people. Ault Foods Ltd., one of the two main industrial dischargers within the drainage basin, is located in Winchester. Municipal sanitary wastewater is gathered in a sanitary sewer system, drained to a pumping

station, pumped to a 5 cell lagoon system, and discharged on a seasonal (2 times per year) basis. The total pumping/treatment system now available for this municipality is relatively new; it was built in the last few years. It is operated by MOE staff.

Alum (for phosphorus precipitation) is added continuously to the lagoon influent. Biological and physical-chemical treatment is provided in four facultative cells. When these cells are ready for discharge, the effluent is passed through an aeration cell. The purpose of this coarse-bubble aeration is to remove any hydrogen sulphide (H_2S) which may be present, especially in effluent discharged during the spring. Prior to installation of the post aeration cell there had been some complaints of sulphide odours from nearby residents.

The WPCP facilities are well maintained and operated. Recorded data show that the effluent quality is well within the C of A limits for this WPCP. Maximum storage volume, available in the four cells is 470 ML. At an average daily flow of 1.8 ML/d, the time between discharges is 261 days.

Alum is used for P precipitation. The dose is near 70 mg/L which is in the normal application range for lagoons at this loading.

Flow data prior to 1990 were considered to be inaccurate and below the actual values. The recorded increase in flow in 1990 is more likely due to the installation of flow metering of total flows as discussed in Section 2.2.

2.3.11 Ault Foods Limited - Winchester

This plant is located in the community of Winchester. It has been gradually enlarged and expanded over the years. It is now one of the largest dairy-related production plants in the province. Milk is received at this plant, and used for production of butter, cheese, milk powder, whey powder, and demineralized whey solids.

Wastewater generated in all the stages of production is collected at a common sump, and is pumped to a WPCP situated on a site located to the north of the Ault production facility. Current wastewater flow is in the range of 0.8 - 0.9 ML/d.

The wastewater characteristics of this industry differ significantly from domestic wastewater. The ten WPCPs described earlier in this brief all receive domestic wastewater with BOD_5 and SS concentrations of 100 - 300 mg/L. However, wastewater from the Ault production plant has BOD_5 and SS concentrations around 2,500 and 1,000 mg/L, respectively. It also carries relatively high concentrations of phosphorus (P) and other dissolved minerals. In terms of BOD_5, the discharge from this production plant is roughly equivalent to domestic discharge from a town of 25,000 people.

At the present time, the wastewater is treated in a biological treatment system and then stored in on-site lagoons for annual discharge in the spring. The required storage capacity is large.

The present C of A specifies the following effluent criteria:

BOD_5 = 5 mg/L

SS	=	15 mg/L
TP	=	1.0 mg/L
NH_3-N	=	5.0 mg/L

The present treatment train includes:

- one wastewater wet-well/pumping station
- two aerated cells in parallel
- one aeration basin to receive effluent from both aeration cells
- one secondary clarifier, with sludge recycle to the aeration basin
- three effluent storage lagoons (#4, #7, #8)
- one effluent storage/polishing treatment lagoon (#6)
- two sludge storage lagoons

Phosphorus removal is achieved by alum addition to the activated sludge treatment system.

The degree of treatment obtained in the WPCP is high. Even though the degree of treatment was relatively high compared to a municipality, Ault Foods was unable to comply with all of its C of A requirements in recent years. The most recent discharge (spring of 1991) had the following effluent characteristics:

BOD_5	=	9 mg/L
SS	=	28 mg/L
TP	=	0.94 mg/L
NH_3-N	=	2.9 mg/L

Ault Foods Ltd. is now preparing for an expansion of their production facilities and for a commensurate upgrading of the WPCP including consideration of in-plant load reduction and effluent reuse. An Environmental Study Report (ESR) is being prepared. It is anticipated that hydraulic loading will increase from 328 ML to 455 ML per year (0.90 to 1.25 ML/d).

2.3.12 Nestle Enterprises Ltd. - Chesterville

This plant is situated in Chesterville. A variety of food and beverage products are made at this plant but instant coffee is the main product.

A substantial amount of wastewater is produced in the plant. It is collected in a common sump and discharged to a WPCP located about 0.5 km from the plant. Daily wastewater flow is near 0.8 ML/d (200 ML/yr). It is frequently highly coloured due to coffee bean extract. The BOD_5 is high with an average value near 1,500 mg/L.

The WPCP treatment train includes the following process units:

- a coarse screen in the influent sewer
- wastewater pumping, to lift it into the treatment (and storage) units
- aerated equalization
- activated sludge treatment
- tertiary treatment/storage in an on-site lagoon

- discharge during all but four summer months
- sludge storage cell

Discharge to the receiving stream is allowed from October 15 to June 15, or for a period of 240 days per year. The average daily flow during discharge is approximately 0.83 ML/d. This WPCP is the only point source in this watershed that is not limited to seasonal or annual discharge.

A recent report dealing with the WPCP lists the capacity of the polishing/storage lagoon as 57,000 m^3. However, to control weed growth and to protect the lagoon bottom, it is necessary to retain some water in the lagoons at all times. For practical purposes, the available storage capacity is therefore less than 57,000 m^3. For discussion purposes an available volume of 40,000 m^3 will be used. A reduced wastewater flow or enlarged storage lagoon volume would be required to comply with the C of A allowance of 240 days for discharge.

This storage capacity implies that summer flows should not exceed 40,000 m^3/4 months, or 10 ML/mo (compared to 16.7 ML/mo indicated by the above flow of 200 ML/yr).

Effluent requirements for this WPCP are:

BOD_5	=	15 mg/L
SS	=	15 mg/L
TP	=	1 mg/L

Even though the WPCP facilities have demonstrated a very high degree of treatment during the last few years, they have not been able to meet the stringent criteria in all cases. MOE published data for 1988 and for 1989 indicated the average effluent quality (all results in mg/L) in the table below.

Parameter	1988	1989
BOD_5	17	23
SS	17	35
TP	1.3	0.8

Steps are now being taken by Nestle to correct limiting factors at this WPCP.

2.4 OTHER SOURCES OF POLLUTION IN THE SOUTH NATION WATERSHED

Other potential point sources of contaminants to the South Nation River system have also been identified. Figure 2-1 indicates the locations of these potential sources, which include stormwater outfalls, landfill sites, and commercial establishments. Although these are not

specifically included in the TOR and will not be dealt with directly as part of this study, it is important to be cognizant of the potential for discharges from these sources.

2.5 POLLUTANT LOADINGS

The loads of nitrogen and phosphorus from municipal effluents can be reasonably assessed from the available data. It is necessary to make assumptions where data are lacking. The assumptions are described as follows.

Lagoon discharge volumes during each month were not always available. In this case it was assumed that the volume accumulated during the previous 6 or 12 months (depending on whether lagoon discharge is annual or seasonal) was discharged during a specified discharge period. Furthermore if the C of A specified that a discharge could occur during, for instance, 2 weeks in April and 3 weeks during May that 2/5ths of the volume to be discharged was discharged during April and the remainder was discharged during May. Similarly if effluent concentration data were unavailable for a portion of the discharge period, the average concentration based on the available data was used.

This approach was used to fill in gaps in data to calculate the monthly effluent loadings (for the months during which discharge occurred) that were discharged from each lagoon. Results are presented in Appendix E. Further assumptions are indicated in the notes accompanying each table in Appendix E. Annual loadings for each point source were also calculated. The nitrogen data for lagoon effluents were the least complete data set.

Only the years 1986 through 1990 were examined because river concentrations have changed significantly during the last 12 years (as discussed in Chapter 4) and lagoon effluent quality information was generally available for the years 1986 through 1990.

2.6 FUTURE EXPANSION IN THE WATERSHED

2.6.1 Population Estimates

The planning period for the study has been established as 20 years. Future populations in the communities have usually been estimated from projections made by the communities themselves and consultant reports on expected population growth. In the absence of reports on growth for a community, a simple arithmetic rate of growth was assumed. Population information and the projected populations for the communities are given in Table 2-7. Only populations of the urban centres are included in this table.

Population growth is dependant on many factors as noted in the Municipal Planning Consultants (1981) report. Their estimates were based in part on historical growth rates and partially on other factors. It was noted that population growth in the late 1970s was generally rapid and was not expected to be maintained. The growth rate adjustment factors were not clearly identified in the report but were based on a statistical review of trends.

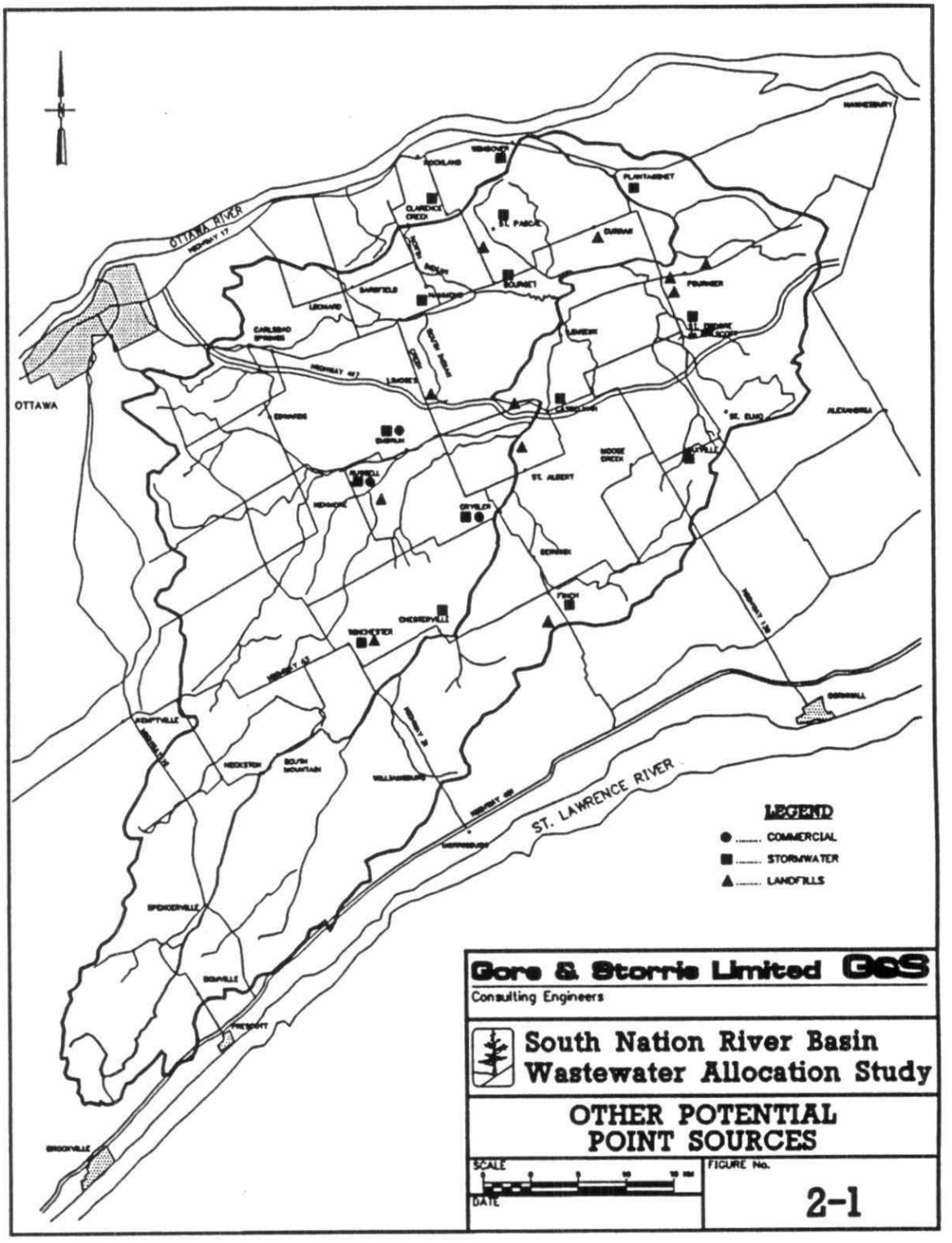

Figure 2-1 Other Potential Pollution Sources in the South Nation Watershed

Table 2-7 Population Projections

		1991				Official Plan		
Municipality	1976[1]	Actual[12]	Proj.[1]	Dif.[8]	2001[1]	(Year)	2011	Growth
Bourget	949		1297		1437		1577	14/yr
Casselman	1425	2100	2025	75	2207		6368	5%/yr[13]
Chesterville	1325	1500	1657	-157	1735		1656	7.8/yr
Crysler	490	1000[10]					1500	2%/yr
Embrun	1763	3500	2406	1094	2666	9399 (2007)	10874	461/yr
Fournier	225	230 (1990)				242[6] (2011)	349[9]	2%/yr
Maxville	850	800	830	-30	835		810	0.5/yr
Moose Creek	382	450 (1989)				560[2] (1999)	696[2]	2%/yr
Plantagenet	920	900	1060	-160	1131	1120[5]	1098	1%/yr
Russell	690	1800	1170	630	1275	5483 (2007)	6404	230/yr
Spencerville	434	350[12]					475	1%/yr
St. Albert	259						948[7]	7.6%/yr
St. Isidore	690	700	849	-149	903	990 (1998)	1040[4]	2%/yr
Williamsburg	407					485[3] (2011)	485	0.5%/yr
Winchester	1745	2200	2525	-325	2746		2642	22.1/yr

1. 1991 and 2001 population projections from Municipal Planning Consultants (1981).
2. ESR for Moose Creek. Geometric growth rate of 2% per annum.
3. Official Plan for the township projects growth rate of 0.5% per annum.
4. Official Plan for the village projects an average growth rate of 2% per year.
5. The Official Plan projects 1% (geometric) as a reasonable growth rate.
6. The Official Plan for the township of Plantagenet reports 0.25% per year as a reasonable growth rate.
7. The Official Plan notes that an arithmetic growth rate of 7.6% per year described past growth.
8. Difference between Municipal Planning Consultants (1981) 1991 population projection and actual population.
9. Kostuch (1990a) used 2% (geometric) as the design number based on discussions with the hamlet.
10. Kostuch reports the existing population equivalent as 1000.
11. MOE (1991). Personal communication.

Notes for Table 2-7 continued.

12. Current population estimate was obtained from the village.
13. Based on a 25 year forecast of over 8000 persons.

Municipal Planning Consultants (1981) population projections were used as the basis for determining populations in the year 2011 for most of the communities. The growth rate was calculated from the projected populations in this report in 1991 and 2001 on an arithmetic basis. This growth rate was applied to the existing population in 1991 to determine the population in 2011. This report was the most comprehensive study of population growth for the whole region and it indicated that growth rates were expected to generally decline.

Therefore an arithmetic as opposed to a geometric growth rate appears to be more reasonable while maintaining a conservative estimate of growth. The communities for which this procedure was used were: Chesterville, Maxville, and Winchester. This procedure was also used for Bourget except that the existing urban population in 1991 for this village was unavailable; therefore the projected population in 1991 given by Municipal Planning Consultants (1981) was used as the base 1991 population to forecast the 2011 population according to the procedure described above.

Official Plans and/or Environmental Study Reports (ESR) were also available for many of the communities. Growth rates projected in these documents were usually higher than growth rates given in the Municipal Planning Consultants (1981) report. Embrun and Russell have the most ambitious expansion plans. Their Official Plan indicated the populations in Table 2-7 for these two towns. The projections for 2011 were obtained by calculating the arithmetic growth rates from the existing 1991 and the year 2007 projections and using this rate to obtain the 2011 forecasts. The Official Plan for Fournier noted that growth in the township was expected to be 1% per annum however, Kostuch (1990a) used a design figure of 2% per annum based on discussion with the village. The latter was chosen as a more conservative figure for the purposes of this study. Kostuch (1990b) reported a design geometric growth rate of 2% for Crysler. Population projections for Casselman were based on estimates made by the municipality's officials.

Notes for the table include the growth rates that were used for the other communities based on Official Plans or ESR reports. The urban population in 1976 for each community taken from the Municipal Consultants (1981) report was used as the base population when this information for 1991 was unavailable. When the Official Plans reported growth rates for the township as a whole but not specifically for the hamlet or village, it was assumed that the village or hamlet would grow at the same rate as the township.

The Official Plan for Plantagenet suggested 1% per annum as a realistic growth rate. Growth in the village of Spencerville was not specifically described in any reports. A median geometric growth rate of 1%/yr was assumed for this community.

Note that in some cases there are discrepancies between the existing populations connected to the sewer system and the urban populations in 1991. Population growth is based on the urban population and it is assumed that the whole urban population will eventually be connected to the sewer system. In summary, the growth anticipated in these communities will almost double the population in the region over the next 20 years.

2.6.2 *Wastewater Flowrates*

Existing per capita wastewater flowrates vary considerably from community to community based on the state of repair of the existing system and illegal connections. The range of flows was from 210 to 1060 L/cap/d as discussed above. Communities with newer sewer systems had rates in the range of 300-400 L/cap/d. The future flow conditions for communities that presently have a sewer system were estimated by adding a flowrate of 400 L/cap/d for the additional population to the existing flow. Metcalf & Eddy (1991) report that domestic water use in the U.S. averages around 230 L/cap/d (range: 152-493 L/cap/d). The additional future population will be serviced by new sewers. Using a value of 400 L/cap/d reflects the fact that the new systems will be fairly tight and provides an allowance for I/I and some system deterioration.

Table 2-8 lists the expected daily flows and annual volumes of wastewater from all communities. The industrial sources are also included. The 1990 flowrates were used for the existing flowrate. Most of the communities had flow metering in place during 1990. The population connected to the existing system is not the same as the population of the village or town at present as noted above. Additional future population includes new residents of the community and those who are not currently connected to the sewer system.

The future flow for Maxville is somewhat anomalous compared to the other flows considering the population data. This is due to the observed low flowrate for this facility in 1990 which largely comprised the future flowrate since Maxville is not expected to experience a significant population change.

2.7 *DESIGN EFFLUENT QUALITY FOR WASTEWATER TREATMENT PROCESSES*

Historical data on municipal lagoons effluent quality were reported in Sections 2.2 and 2.3. These data were fairly limited; there were not any measurements of H_2S. A review of more comprehensive data in other sources (MOE, 1990 and Environment Canada, 1985) for effluents from municipal lagoons yielded clear trends in the variability of BOD_5, NH_3-N and H_2S as a function of temperature, ice cover and winter conditions.

Winter conditions lead to an increase in ammonia and hydrogen sulphide as dissolved oxygen (DO) and temperature fall. Peak values of the pollutants occur in mid-February to mid-March for continuous discharge facultative lagoons. Only lightly loaded lagoons can be expected to maintain a DO above 1 mg/L under the ice cover which prevents the formation of H_2S. If a system has a mechanical treatment facility (e.g., extended aeration activated sludge) that discharges to a storage lagoon, DO generally will be maintained above 1 mg/L during winter conditions and H_2S formation will not be a problem.

Spring concentrations for ammonia and hydrogen sulphide drop rapidly as ice melts and temperatures rise. H_2S concentrations dropped to 0.1 mg/L or less by mid-April for most lagoons. Ammonia nitrogen concentrations did not drop as rapidly as hydrogen sulphide concentrations. Ammonia returned to "warm weather" concentrations by late April to mid-May.

Table 2-8 Wastewater Flowrates

Municipality	Connected Pop. 1991[1]	Flow 1991 m^3/d	Annual Volume 1991 1000 m^3	Pop. 2011	Flow 2011 m^3/d	Annual Volume 2011 1000 m^3
Bourget	-	-	-	1577	631	230.2
Casselman	2400	1000	365.0	6368	2587	944.3
Chesterville	1500	811	296.0	1656	873	318.8
Crysler	-	-	-	1500	600	219.0
Embrun	3500	1300[2]	474.5	10874	4249	1551.1
Fournier	-	-	-	349	139	50.9
Maxville	800	164	59.9	810	168	61.3
Moose Creek	-	-	-	696	278	101.6
Plantagenet	900	617	225.2	1098	696	254.1
Russell	1800	529	193.1	6404	2370	865.3
Spencerville	350	140[2]	51.1	475	190	69.4
St. Albert	-	-	-	948	379	138.4
St. Isidore	700	348	127.0	1040	484	176.7
Williamsburg	439[3]	176[2]	64.1	485	194	70.8
Winchester	2200	1725	629.6	2642	1965	717.2
Industry						
Ault	-	531[4]	193.8	-	1246[4]	455.0
Nestle	-	619[5]	225.9	-	619[5]	225.9
St. Albert's Cheese	-	-	-	-	62[6]	22.6

1. Personal communication (MOE, 1991)
2. Estimated flow.
3. No population or flow information was available. The 1976 census population given by Municipal Planning Consultants (1981) was increased by the 0.05% geometric growth rate given in Table 2-7 for 15 years to arrive at the 1991 population estimate.
4. Ault's 1991 daily flow is the average of the annual volume for 1988-89 data divided by 365. The future flow was taken as the upper flow projection in a recently prepared ESR for this industry.
5. Nestle's average daily flow is the average of the annual volume for 1986-90 data divided by 365. There is no expansion projected by Nestle.
6. Acres International (1990).

For 16 of 25 lagoons studied, DO was greater than 4.0 mg/L and H_2S was less than 1.0 mg/L within two weeks from the time that ice started to melt (i.e., greater than 5% of the lagoon area is free of ice). All systems had "sweetened" within eight weeks after the time ice started to melt.

Oxygen supplied to a facultative lagoon by diffused aeration equipment should be able to provide sufficient DO to prevent H_2S formation or to rapidly oxidize it prior to discharge during the spring. When ice cover is significant the mechanism of H_2S removal is oxidation, not stripping.

A similar situation exists regarding ammonia. The rate of ammonia production during the winter will be reduced if aerobic conditions are maintained. However, since the biological processes are so strongly dependent on temperature, some NH_3-N will be present regardless of the DO when the temperature falls below 5°C. Subsequent removal of NH_3-N produced during the winter occurs more slowly than H_2S removal due to the effect of temperature. During the summer, ammonia removal is due to oxidation (both chemical and biological), uptake by algae and other microorganisms and stripping with elevated pH and supersaturated oxygen conditions.

Based on the review of the above lagoon data sources and requirements from MOE to use C of A effluent values when specified, the parameters of interest for lagoons and various treatment processes are given in Table 2-9. The New Hamburg and Sutton processes are advanced treatment options that incorporate lagoons which increase their potential as upgrades to many of the existing facilities.

Table 2-9 Suggested Effluent Concentrations for Various Wastewater Treatment Processes

	Parameter Concentration, mg/L					
Treatment Process	BOD_5	SS	TP	NH_3	H_2S	pH[2]
Lagoon, Spring Discharge	30	20	1[1]	April- 14 May- 6	April- 3 May- 1	8.5
Lagoon, Fall Discharge	20	40[3]	1[1]	2	0	8.5
New Hamburg	2.4	5	0.4	4	0	8.5
Extended Aeration Activated Sludge	15	15	1	Winter- 5 Summer- 1.2	0	6-8
Sutton Process[3]	5	10	0.5-1.0	2	0	8.5

1. With continuous alum addition.
2. pH is on the pH scale.
3. The Sutton process is under review. See text.

The New Hamburg process consists of an aerated facultative lagoon followed by waste stabilization ponds for effluent polishing and seasonal storage. The effluent from the polishing pond is pumped to a battery of intermittent slow sand filters. Besides providing for

the removal of SS and associated BOD, nitrification and denitrification occur in the filter beds as a result of the intermittent loading operation and biological film which develops in the filter bed.

The Sutton process consists of an extended aeration activated sludge plant where the effluent from the clarifier after the extended aeration basin is discharged to a polishing lagoon. Waste sludge is also discharged to the polishing pond for sludge reduction and to promote denitrification of the nitrified effluent from the clarifier. Retention times are 2-3 months in the polishing pond. The Sutton process appeared to be very promising from results available at the beginning of this study; however, recent data has put the process under review (MOE, 1992). **Therefore this process will not be considered in the modelling exercises.**

For mechanical treatment of sewage small package plants designed to operate in the extended aeration mode (which will result in nitrification) are the most appropriate option for small communities.

Lagoon effluent pHs can be greater than 8.5 in the spring (March - May) although winter values for southwestern Ontario lagoons were in the range of 7.4-7.6 (MOE, 1980). Algae activity is the largest influence on pH fluctuation. pH variation in maturation ponds can range from 6.5-10.5 (Metcalf & Eddy, 1991). The higher pHs generally occur during the summer months when algae activity is highest. Since discharges will be confined to the spring and fall, a pH of 8.5 was chosen as a reasonable value for effluent from the New Hamburg and Sutton processes. An extended aeration process should operate in the pH range of 6-8 to maintain treatment efficiency.

The effluent concentration allowances for Ault and Nestle are given in Sections 2.3.11 and 2.3.12, respectively. For the Ault industry the C of A specifies a NH_{3T}-N concentration of 5 mg/L which will be used in all calculations for this industry. However, there is no current C of A specification for the NH_{3T} concentrations in Nestle's effluent. Nestle's treatment process has an aerated equalization basin, followed by an extended aeration activated sludge process. Clarified effluent from the extended aeration process is treated with alum and polymer before it is pumped to an aerated polishing pond. An estimate of NH_{3T}-N effluent concentration is 5 mg/L. Gore & Storrie Limited has examined recent data for the process which confirms that this assumption is reasonable. It also results in equitable effluent quality for this parameter between the two major industries. Therefore this value will be used in all evaluation and modelling exercises.

There is also no current C of A issued for the St. Albert Cheese industry. The effluent NH_{3T}-N concentration for this industry will be set at 5 mg/L to be consistent with the requirements for the other two industries. Effluent BOD_5 will be arbitrarily set at 15 mg/L which is the current specification for Nestle.

SECTION 3

HYDROLOGICAL REGIMES

HYDROLOGICAL REGIMES

3.1 INTRODUCTION

One of the primary tasks for this study was to establish the hydrologic regime of the South Nation River and its tributaries. Of most interest to the study was the estimation of low flows during the spring and fall, during which time the municipalities in the watershed discharge their lagoons to the receiving waters. The task therefore entailed a low flow frequency analysis to determine the '7Q20' and '30Q20' flows in the receiving streams for each discharge location within the watershed for the spring and fall periods. Due to the lack of flow data at several of the discharge locations, a proration of flow as a linear function of drainage area was required. The low flows estimated in the analysis were used as a basis for modelling of the river and tributaries. The final objective of the river modelling was to determine the assimilative capacity of the receiving water for future discharges. It is therefore important to obtain conservative yet reasonable estimates of low flows.

Due to the intermittent nature of some of the gauging station locations, an approach for the low flow analysis was taken which differs from the conventional method. As a result, details of the analysis are provided herein, to further the understanding of the methodology used.

3.1.1 Stream Gauging Stations

A list of all stream gauging stations within the South Nation River Watershed was obtained from the Water Survey of Canada (WSC). Daily flow records were obtained from the WSC for these stations. The list below includes the station number, location, receiving water river/stream, years of record and overall tributary area. Figure 3-1 shows the locations of the streamflow monitoring stations.

The tributary areas for each site were confirmed with topographical mapping of the study area using 1:250,000 (contour interval: 15.24 m) and 1:50,000 (contour interval: 10 m).

- 02LB005 Plantagenet (S. Nation River) 3,820 km^2
  - RC* = 1963-86
  - MC = 1948-62, 1915-38
  - MS = 1939-47
- 02LB006 Russell (Castor River) 449 km^2
  - RC = 1968-86
  - MS = 1948-67
- 02LB007 Spencerville (S. Nation River) 243 km^2
  - RC = 1960-86
  - MC = 1949-59
  - MS = 1948
- 02LB008 Bourget (Bear Brook) 436 km^2
  - RC = 1976-86

MS = 1955-69, 1950-53
MC = 1949

- 02LB009 Chesterville (S. Nation River) 1,059 km²
  RS = 1985-86
  MS = 1984, 1975-82, 1955-71, 1949-52
  MC = 1972-74
- 02LB013 Casselman (S. Nation River) 2,400 km²
  RC = 1975-86
  MC = 1972-74
- 02LB017 Heckston (N. Br. S. Nation River) 106 km²
  RC = 1977-86
- 02LB018 St. Isidore (W. Br. Scotch River) 100 km²
  RS = 1984-86
  RC = 1979-83
- 02LB020 Kenmore (South Castor River) 201 km²
  RC = 1978-86
- 02LB022 Berwick (Payne River) 150 km²
  RC = 1976-86
- 02LB101 Carlsbad Springs (Bear Brook) 65 km²
  RC = 1976-78
  RS = 1975

* R = recording gauge
M = Manual gauge
C = Continuous operation
S = Seasonal operation

3.2 *THE TRADITIONAL APPROACH*

The traditional approach was utilized in the first attempt to estimate low flows. The methodology used is briefly described here.

There were 8 flow gauging stations within the South Nation River watershed for which adequate daily flow data were available. These are:

Station No.	Location
02LB005	South Nation River at Plantagenet
02LB006	Castor River at Russell
02LB007	South Nation River at Spencerville
02LB008	Bear Brook at Bourget
02LB013	South Nation River at Casselman
02LB017	North Branch South Nation River at Heckston
02LB020	South Castor River at Kenmore
02LB022	Payne River at Berwick

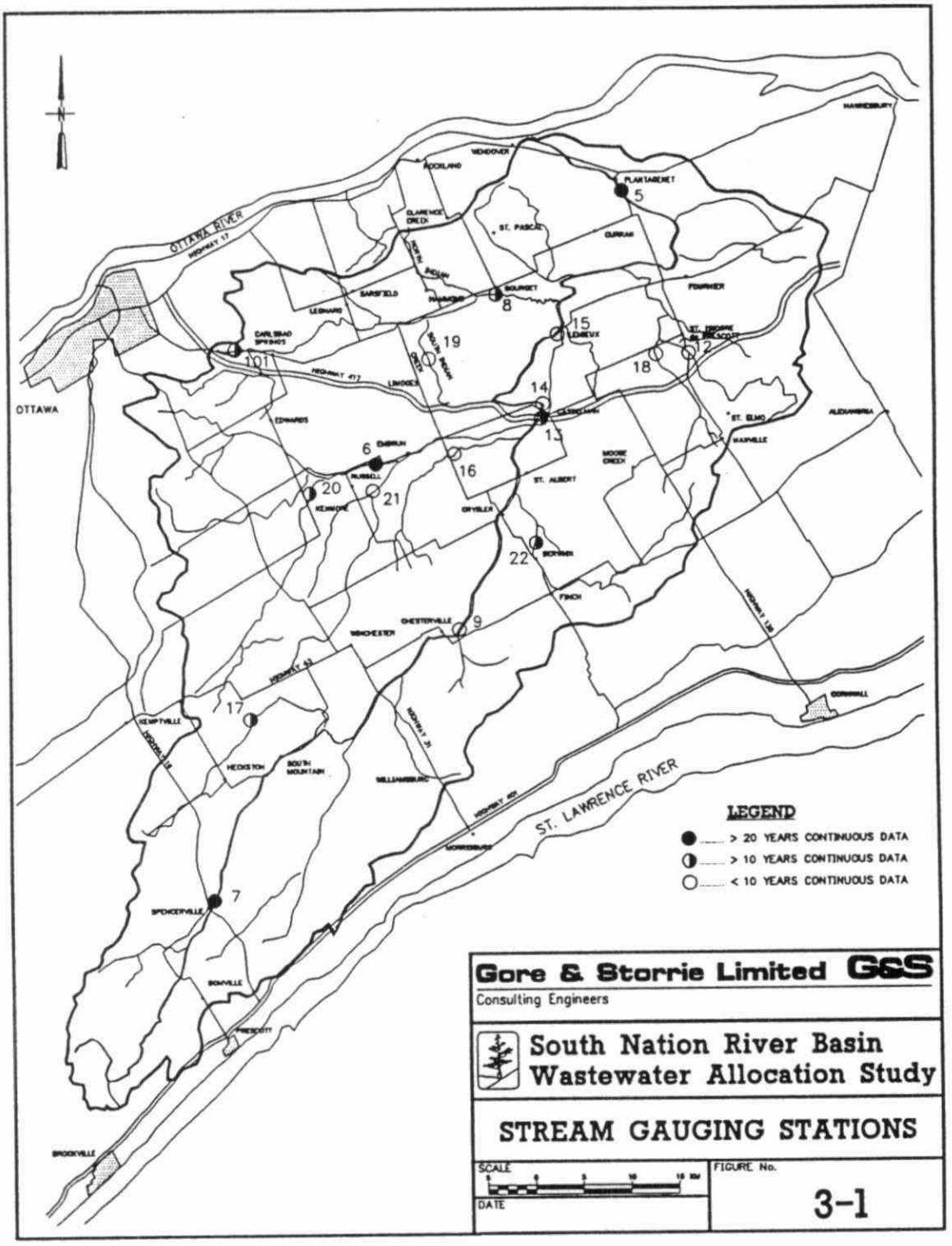

Figure 3-1 Stream Gauging Stations in the South Nation Watershed

Data were also available at the following stations, but were not used, due to insufficient data.

Station No.	Location
02LB009	South Nation River at Chesterville
02LB018	West Branch Scotch River at St. Isidore
02LB101	Bear Brook at Carlsbad Springs

The objective of the analysis was to compute 7 and 30 day low flows with 20-year return periods (7Q20s and 30Q20s) for each station, obtain relationships between low flow and drainage areas, and obtain low flow estimates for the discharge locations of interest which do not have measured flow data.

3.2.1 Methodology

A FORTRAN program was written that scanned the WSC data files and computed 7 and 30 day average low flows for March-April and September-October for each year where data existed. For the spring period, the program would extract all daily flow data between March and April of each year, and compute the average flow over all 7 day and 30 day spans in the 2 month period. The program would then extract the lowest of these averages over the entire period. For each year, therefore minimum 7 and 30 day average flows (1 value for each) would be obtained. The same procedure was followed for the September-October period. Thus for each year, 4 data points were obtained; the 7 and 30 day minimum flows for March-April, and for September-October.

Each data series was analyzed for its suitability for frequency analysis. A flood frequency analysis software package was used to assist in the analysis. Since frequency analysis is probabilistic, the data series must be random, and individual events must be independent of each other. The data series should also be homogeneous and stationary. All but 3 of the data sets passed the checks. The data sets rejected for frequency analysis (all on the basis of a lack of independence between data values) were:

Station 02LB007 (7 day low flows, fall)
Station 02LB008 (7 day low flows, spring)
Station 02LB017 (30 day low flows, spring)

For the remaining data sets, a FORTRAN program was written to rank the data (smallest data points ranked highest), and compute return period, probability and the corresponding standard normal variate (z) for each data point. The flows were then plotted versus 'z' for each data set, and the quantile estimate of low flow corresponding to a return period of 20 years ($p = 0.05$, $z = 1.645$) was read from the graph. An alternate approach for the estimation of the quantiles would have been the use of a program developed by Environment Canada (Condie and Cheng, 1983),

called LFA (Low Flow Frequency Analysis package). Similar results would be expected.

Linear regression analysis was performed between the low flow quantile estimates and drainage areas, for the 7Q20s (spring and fall) and the 30Q20s (spring and fall). Low flow estimates for all the ungauged discharge locations in the watershed were computed from the resultant regression equations, knowing the drainage areas tributary to each location.

3.2.2 *Results*

Results of this analysis are not presented in detail herein, as they were not used in the study. The major problem that resulted from the analysis was that for some stations, the predicted 7Q20s and 30Q20s were found to be 0 m^3/s, especially during the fall period. This occurs when there are 7 or 30 day spans where average daily flows of 0 m^3/s were measured by WSC. When the minimum flow is extracted from a series of average flows, where some of these flows are 0 m^3/s, this is the value that becomes the minimum for that year. This situation does occur at several of the gauging stations. Fall 7Q20s of 0 m^3/s were found for St. Isidore, Maxville, Russell, Williamsburg and Spencerville. Fall 30Q20s of 0 m^3/s were found for Maxville and Williamsburg. There were even 7Q20s of 0 m^3/s during the spring period, for Maxville and Williamsburg. If these results were taken at face value, they indicate that there is no assimilative capacity at these locations for any amount of discharge, even at the present levels, let alone future increased discharges. Continusous discharge over 12 months is obviously not feasible.

The results were overly conservative; however, since treatment plant operators would simply not discharge when the receiving stream is dry. They would wait until sufficient flow was observed. Thus, in order not to limit any future development at some of the municipalities on the basis of overly conservative flow estimates, an alternative approach was required.

Another limitation of the conventional approach is that no information is provided regarding the time variability of the river flowrate, and therefore no information on the optimum discharge periods. For the conventional analysis, spring and fall periods encompassing March-April and September-October respectively were used. These are not necessarily the optimum periods for discharge, in terms of the maximum river capacity available for waste assimilation. Thus the alternate approach developed also provides a method of determining 'optimum' discharge periods.

3.3 *ALTERNATIVE APPROACH*

3.3.1 *Introduction*

An important factor in the determination of low flows is the time period, or 'window' used for the computation. The window could have a significant impact on the river capacity, or dilution volume available for a discharge. For example the low flows computed between March and April are likely to be significantly greater than those

computed during May and June. In addition, flows in October and November are expected to be greater than flows in September. If a period of time in the discharge season (spring or fall) can be found during which river volume is maximum, based on a statistical analysis of historical flows, then the optimum window for discharge can be determined. If care is taken to ensure discharges occur during the optimum window, the effective river volume available for wastewater dilution will be increased.

3.3.2 Methodology

In order to determine the optimum time for discharge for the spring and fall seasons, a modified approach was used for the frequency analysis. Rather than computing the minimum 7 day low flow for the spring (March - April) period of each year, the analysis was conducted on a daily basis. Beginning on March 1 of each year, and extending to the end of June (in order to cover the entire spring period), the 7 day and 30 day average flows were computed. Thus, the computations included the average of the flows from March 1 - March 7, March 2 - March 8, up to June 24 - June 30 (a total of 116, 7 day average flows); and from March 1 - March 30, March 2 - March 31, up to June 1 - June 30 (a total of 93, 30 day average flows). A similar set of computations was completed for the fall period, from September 1 to December 31 of each year (to encompass the entire fall period). A total of 418 average flows were obtained for each year. For each of these 418 flows, a frequency analysis was completed over the years of record, to obtain 7Q20s and 30Q20s for each time span. This approach allows for statistical determination of optimum discharge windows, as described below.

As discussed in Section 3.2.1, minimum 7 and 30 day low flows for spring and fall, computed using the conventional approach, were screened for suitability for frequency analysis. A flood frequency analysis software package was used to determine the randomness, independence, homogeneity and stationarity of the data, and, in most cases, the data were acceptable. Only three of the 24 data sets were rejected based on a lack of independence. A similar screening was not conducted for each of the 418 flow data sets computed for each station using the alternative approach due to the onerous time requirements of this task. It was assumed for purposes of the analysis, since the large majority of the original 24 data sets were found to be suitable, that the new data sets were also suitable for frequency analysis.

A FORTRAN program (FLOW2) was written that scanned the WSC data files and computed 7 and 30 day average flows on a daily basis between March 1 and June 30, and September 1 and December 31, for each year. The chief difference between this program and the one written for the traditional analysis is that the minimum 7 day and 30 day average flows are not computed.

The program FLOW2 provided a series of values for both spring and fall periods for each year, rather than a single value. The program produced, for each starting date (March 1 to June 24, and September 1 to December 25), an output file containing two sets of flows; those averaged over the next 7 days and the next 30 days from the starting date. The files for starting dates June 2 and onwards, and December 3 and onwards, only contained 7 day average flows (since June 1-30 and December 2-31

were the last possible time spans for flows averaged over 30 days). These output files became the input for a frequency analysis program developed in-house.

The frequency analysis program fits the flow data to a Gumbel Type III frequency distribution. The three parameters (lower boundary, characteristic drought, and shape parameter) were estimated by the method of smallest observed drought. The Gumbel III distribution was found by Condie and Cheng (1983) to be the most suitable distribution for low flows. The method of smallest observed drought was found to provide acceptable fits on low flow data from 27 out of 38 Canadian rivers tested. The computations in the in-house program are based on the methods used in the Environment Canada program LFA (Condie and Cheng, 1983), and thus the in-house program would provide similar results. The program LFA was not used for the present study since the program requires manual data input via keyboard entry. With 8 flow stations, and 418 flow series per station, manual data input was considered too onerous. The in-house program was designed to automate the frequency analysis procedure.

The data contained in each output file from FLOW2 were read into the frequency analysis program in a batch-wise fashion, the computations were performed, and 20-year quantile estimates were obtained for the 7 and 30 day average flows for each time span.

The next step was to plot daily 7Q20s and 30Q20s versus the starting date of the associated time span at each station. In this way, information on the time variability through the spring and fall seasons of the river flows is obtained. The program references the flows in a 7 day or 30 day period to the starting date of the period. However, the correct actual time occurrence of the flow is on the middle date of the period. For example, the flow in the period March 1-30 should be referenced to the date of Mar. 16; the flow in March 2-31 should be referenced to Mar. 17; etc. Similarly for 7 day flows, 3 days should be added on to the starting date to obtain the correct actual time reference. For the period Mar 1-7, the flow should be referenced to Mar. 4; for the period March 2-8, the flow should be referenced to Mar. 5.

For illustrative purposes, some example plots are provided herein. The dates indicated on the plots are actual time dates. Daily 30Q20s for the spring period are shown as a function of time in Fig. 3-2 for station 02LB005. The figure shows a typical shape for the spring 30Q20s, with a dramatic increase in late March, and a decrease through the month of April.

Using the flow versus time plots, optimum windows for spring and fall discharge could be found. The methodology used was as follows:

- For various flows, the duration of time each flow value is equalled or exceeded was noted. In Fig. 3-2, for example, a 30Q20 of at least 20 m^3/s is maintained between March 18 and April 27, for a total time duration of 40 days. A higher 30Q20 of 60 m^3/s is maintained for a shorter duration, between March 27 and April 15 (19 days). For several other 30Q20 values (every 10 m^3/s), durations were obtained

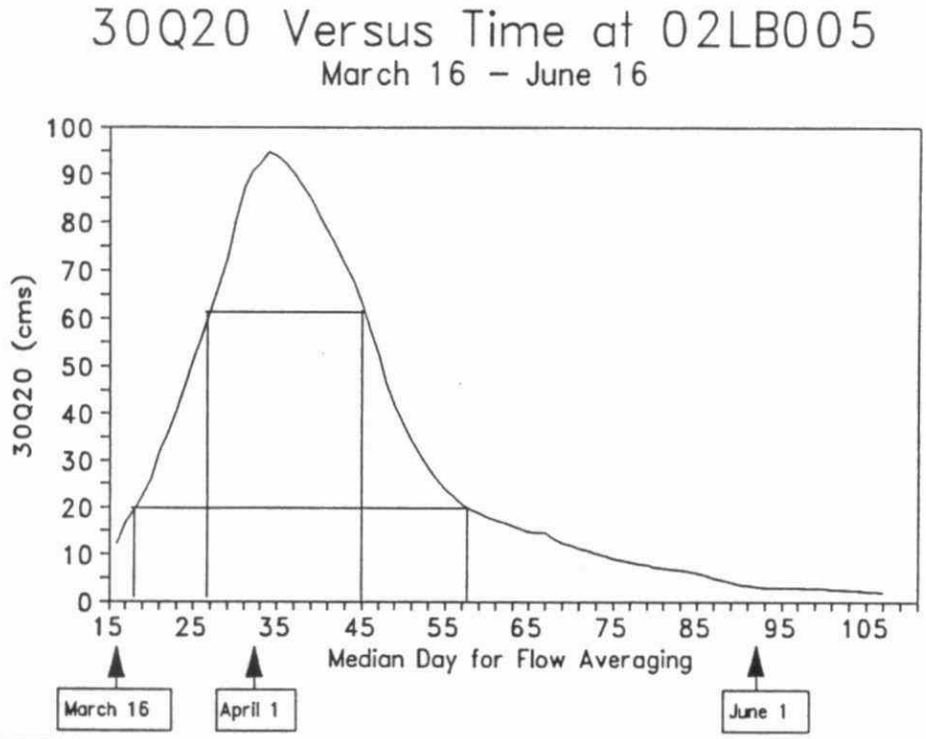

Figure 3-2 Spring 30Q20 Flows at Plantagenet

in a similar manner. From Fig. 3-2, the following flow versus duration data points were obtained:

30Q20 (m^3/s)	Duration (days)
20	40
30	32
40	26
50	22
60	19
70	14
80	11
90	5

- These flow values and durations were then used to compute an 'implied dilution volume' (flow x duration). This represents a conservative estimate of the total river capacity available for waste assimilation.

- Both volume and flow were then plotted versus duration. For the example time series plot shown in Fig. 3-2, a flow and volume versus

duration plot is provided in Fig. 3-3. To illustrate the development of Fig. 3-3, referring back to Fig. 3-2, a 30Q20 of 20 m^3/s is equalled or exceeded between Mar. 18 - Apr. 27 for a total duration of 40 days, and thus the dilution volume is computed as (20 m^3/s x 40 days x 86400 s/d) = 6.9 x 10^6 m^3. This results in the 2 points on the right hand side of the graph in Fig. 3-3. The remaining points were plotted using the same methodology.

- The maximum implied dilution volume, and the corresponding flow and duration were obtained from the plot in Fig. 3-3.

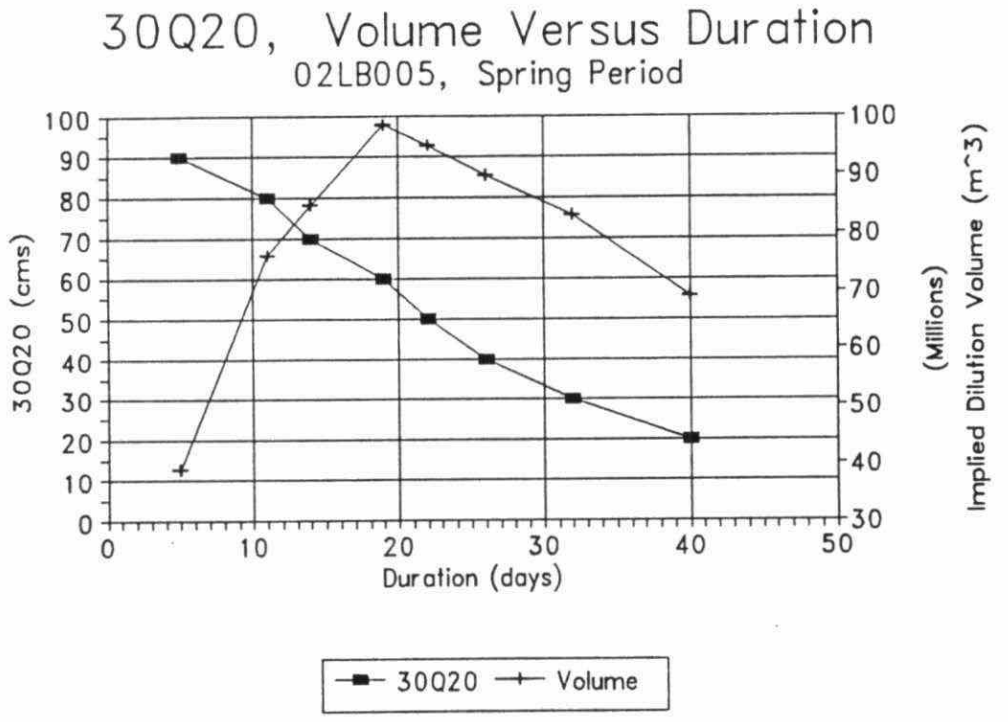

Figure 3-3 Dilution Volume and Duration of Flow

- The optimum discharge window corresponds to the period of maximum dilution volume, since this is the period during which the highest waste volume can be discharged. This optimum window was obtained by examination of the graphs illustrated in Figs. 3-2 and 3-3. Figure 3-3 indicates that the maximum volume occurs at a duration of 19 days, corresponding to a 30Q20 of 60 m^3/s. Referring back to Fig. 3-2, the window during which a flow of 60 m^3/s is equalled or exceeded is from March 27 to April 15. This represents the optimum window for discharge, based on the 30Q20, during the spring period at this station.

- This procedure was repeated for each station, for each 7Q20 and 30Q20 spring and fall flow. Plots of quantile estimates of flow versus time (similar to Fig. 3-2), and flow and volume versus duration (Fig. 3-3) were produced for each station and season, for 7 day and 30 day flows.

The results are described in the following section.

3.3.3 *Results*

The results of the determination of optimum discharge windows are presented in Table 3-1, for spring 30Q20 flows. The procedure to determine discharge windows for this table will be explained in detail. Table 3-1 includes the optimum window, duration and optimum 30Q20 for the spring (March 1 to June 30) period, for each of the 8 gauging stations.

Table 3-1 Results of Optimization of Discharge Window for 30Q20 - Spring

Station	Discharge Window Dates	Duration (days)	30Q20 (m^3/s)
02LB005	Mar 27 - Apr 15	19	60
02LB006	Mar 28 - Apr 16	19	6
02LB007	Mar 21 - Apr 21	31	3
02LB008	Mar 19 - Apr 17	29	6
02LB013	Mar 16 - Apr 21	35	20
02LB017	Mar 16 - Apr 24	39	0.6
02LB020	Mar 16 - Apr 22	37	1.5
02LB022	Mar 20 - Apr 21	33	1

Minimum Window:	Mar 28 - Apr 15
Average Window:	Mar 20 - Apr 20
Maximum Window:	Mar 16 - Apr 24

As expected, the optimum discharge windows do not vary significantly from station to station. However, in order to extrapolate the flow quantile estimates to ungauged discharge locations, some decision had to be made regarding the optimum window to use, as the window has a significant influence on the flow estimate. For the present study, minimum, average and maximum discharge windows were determined. The minimum window was obtained by taking the latest start date (March 28) and the earliest end date (April 15) of the optimum windows for each of the 8 stations. The

average window was found from the average start date (March 20) and average end date (April 20). The maximum window was obtained from the earliest start date (March 16) and latest end date (April 24). These windows are shown beneath Table 3-1.

For each of the minimum, average and maximum discharge windows determined for this case, the 30Q20s for station 02LB005 were determined from examination of Fig. 3-2. The minimum discharge window provided the largest flow quantile value, while the maximum window provided the lowest (or worst case, and most conservative) flow quantile value. To illustrate, refer to Fig. 3-4, which shows the same data as Fig. 3-2. During the average discharge window (March 20 to April 20), the 30Q20 is always greater than or equal to 26 m^3/s. This represents the 'minimum daily 30Q20' for the average spring discharge window. Flow estimates corresponding to the minimum and maximum spring discharge windows of 57 and 12 m^3/s respectively were obtained for station 02LB005 in the same manner. These are representative of the values that were subsequently used in the regression analysis for the determination of flow versus drainage area.

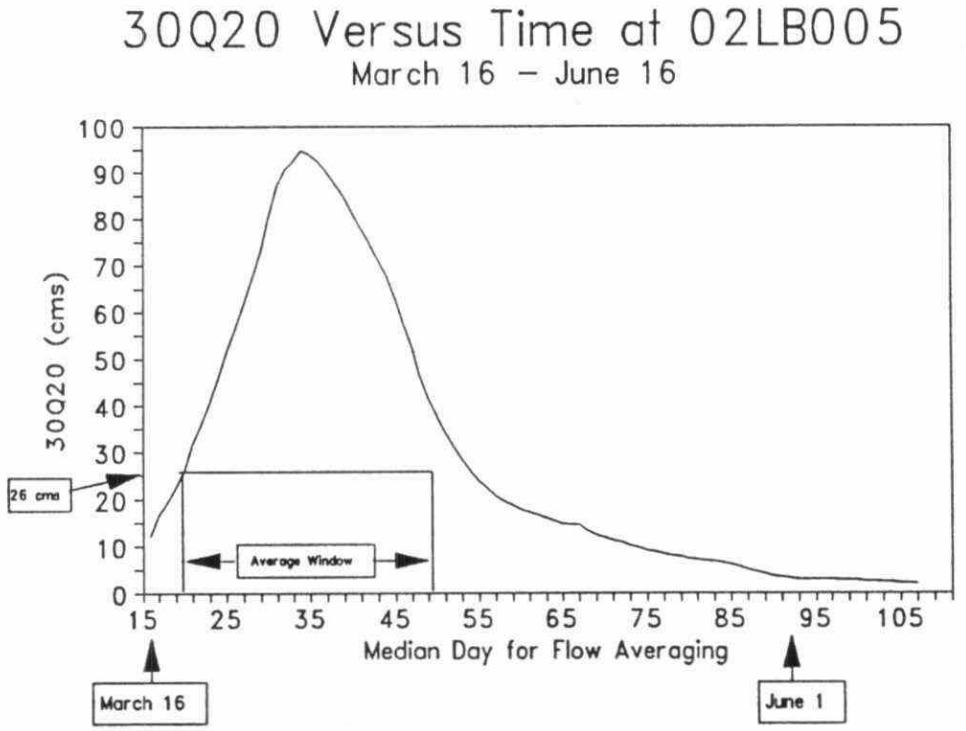

Figure 3-4 Average Spring 30Q20 Window at Plantagenet

The procedure outlined above was repeated for each gauging station, for the estimation of the minimum daily 7Q20s and 30Q20s for the average, minimum and maximum discharge windows during the spring and fall periods. Results for the other conditions are given in Tables 3-2 - 3-4.

Table 3-2 Results of Optimization of Discharge Window for 7Q20 - Spring

Station	Discharge Window Dates	Duration (days)	7Q20 (m^3/s)
02LB005	Mar 30 - Apr 13	15	30
02LB006	Mar 22- May 13	55	1
02LB007	Mar 25 - May 8	44	1
02LB008	Mar 24 - Apr 14	20	4
02LB013	Mar 23 - Apr 23	31	10
02LB017	Mar 23 - Apr 23	35	0.5
02LB020	Mar 24 - Apr 22	32	1
02LB022	Mar 24 - Apr 29	45	0.4

Minimum Window: Mar 30 - Apr 13
Average Window: Mar 25 - Apr 26
Maximum Window: Mar 22 - May 13

Table 3-3 Results of Optimization of Discharge Window for 30Q20 - Fall

Station	Discharge Window Dates	Duration (days)	30Q20 (m^3/s)
02LB005	Nov 13 - Dec 17	34	1.4
02LB006	Nov 14 - Dec 17	33	0.6
02LB007	Nov 24 - Dec 17	24	0.2
02LB008	Nov 5 - Dec 17	41	0.5
02LB013	Nov 18 - Dec 17	29	1.4
02LB017	Nov 20 - Dec 17	26	0.05
02LB020	Sept 16 - Dec 14	90	0.08
02LB022	Nov 16 - Dec 15	29	0.2

Minimum Window: Nov 24 - Dec 15 (excluding 02LB020)
Average Window: Nov 16 - Dec 17 (excluding 02LB020)
Maximum Window: Nov 5 - Dec 17 (excluding 02LB020)

Table 3-4 Results of Optimization of Discharge Window for 7Q20 - Fall

Station	Discharge Window Dates	Duration (days)	7Q20 (m^3/s)
02LB005	Nov 8 - Dec 16	43	0.4
02LB006	Nov 19 - Dec 15	26	0.6
02LB007	Nov 25 - Dec 14	21	0.12
02LB008	Nov 10 - Dec 29	68	0.2
02LB013	Nov 16 - Dec 14	32	1.0
02LB017	Nov 27 - Dec 28	32	0.03
02LB020	Sep 24 - Nov 28	89	0.08
02LB022	Nov 10 - Dec 18	39	0.1

Minimum Window: Nov 27 - Dec 14 (excluding 02LB020)
Average Window: Nov 16 - Dec 19 (excluding 02LB020)
Maximum Window: Nov 8 - Dec 28 (excluding 02LB020)

For each of the minimum, average and maximum discharge windows, the 7Q20s and 30Q20s were determined from examination of the flow versus time plots. The results are presented in Tables 3-5 - 3-7 along with the drainage areas for the stations.

Table 3-5 Spring 30Q20 Minimum Flows at Gauged Stations

Station	Drainage Area (km^2)	Minimum Daily 30Q20 (m^3/s)		
		Min. Window	Ave. Window	Max. Window
02LB005	3820	57	26	12
02LB006	459	6.2	1.6	1.2
02LB007	243	3.7	2.3	1.5
02LB008	436	7	5	3
02LB013	2400	24	20	19
02LB017	106	0.68	0.67	0.6
02LB020	201	2	1.65	1.4
02LB022	150	1.09	1.0	0.86

Table 3-6 Spring 7Q20 Minimum Flows at Gauged Stations

Station	Drainage Area (km^2)	Minimum Daily 7Q20 (m^3/s)		
		Min. Window	Ave. Window	Max. Window
02LB005	3820	30	13	6
02LB006	459	1.4	1.2	1.0
02LB007	243	1.9	1.1	0.8
02LB008	436	4.6	1.5	0.8
02LB013	2400	18.5	7	4
02LB017	106	0.67	0.35	0.15
02LB020	201	1.75	0.8	0.4
02LB022	150	0.9	0.4	0.25

Table 3-7 Fall 30Q20 Minimum Flows at Gauged Stations

Station	Drainage Area (km^2)	Minimum Daily 30Q20 (m^3/s)		
		Min. Window	Ave. Window	Max. Window
02LB005	3820	1.7	1.54	1.03
02LB006	459	0.68	0.64	0.21
02LB007	243	0.2	0.12	0.04
02LB008	436	0.53	0.53	0.44
02LB013	2400	1.44	1.29	0.34
02LB017	106	0.05	0.037	0.0
02LB020	201	0.08	0.08	0.08
02LB022	150	0.21	0.155	0.115

The 30Q20 flows for gauged stations that were common between this study and the study of Cumming Cockburn Ltd. (1989) showed reasonable agreement considering differences in the approaches. The latter study examined the 30Q20 flows for each month. In the current study none of the windows are exactly confined to a given month but some of the windows do approximately span a month and it is possible to make an approximate comparison.

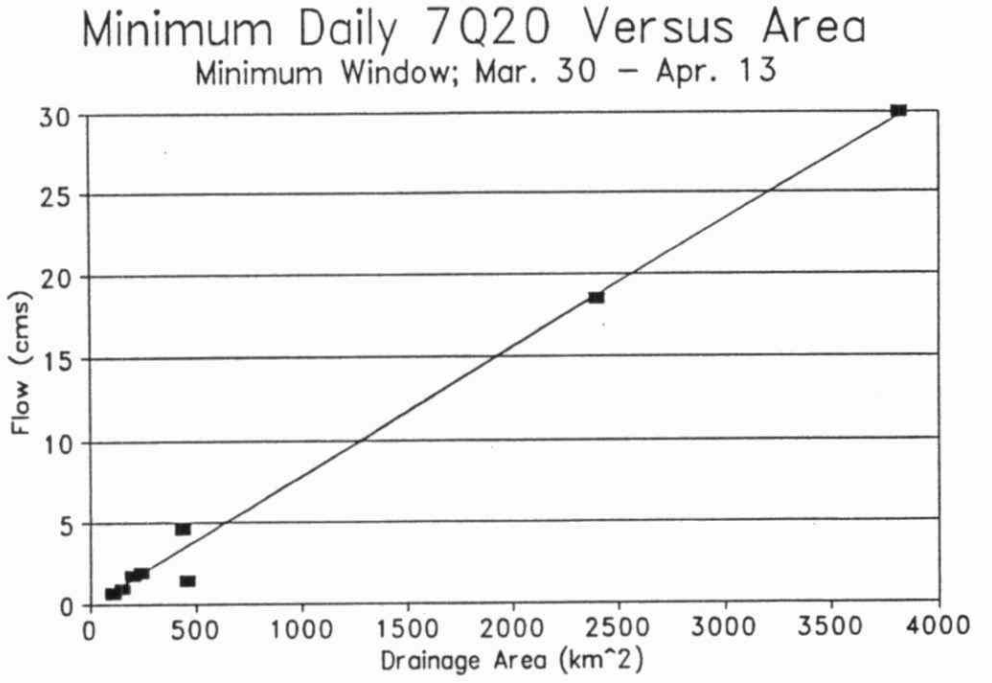

Figure 3-5 Spring 7Q20 Minimum Window Flow - Drainage Area Correlation

Linear regression analyses were performed between the low flow quantile estimates (determined as discussed above) and drainage areas to obtain flow estimates for the ungauged discharge locations. The regression equations for flow as a function of drainage area (A_D) and the associated discharge window durations are listed below. The data correlated very well. Typical plots of the data are given in Figs. 3-5 and 3-6.

In the equations below, Q is in m^3/s and A_D is in km^2.

Spring 7Q20

Minimum Window (Mar. 30 - Apr. 13; 15 days):

$$Q = 7.79 \times 10^{-3}(A_D) \qquad r = 0.996$$

Average Window (Mar. 25 - Apr 26; 33 days):

$$Q = 3.27 \times 10^{-3}(A_D) \qquad r = 0.996$$

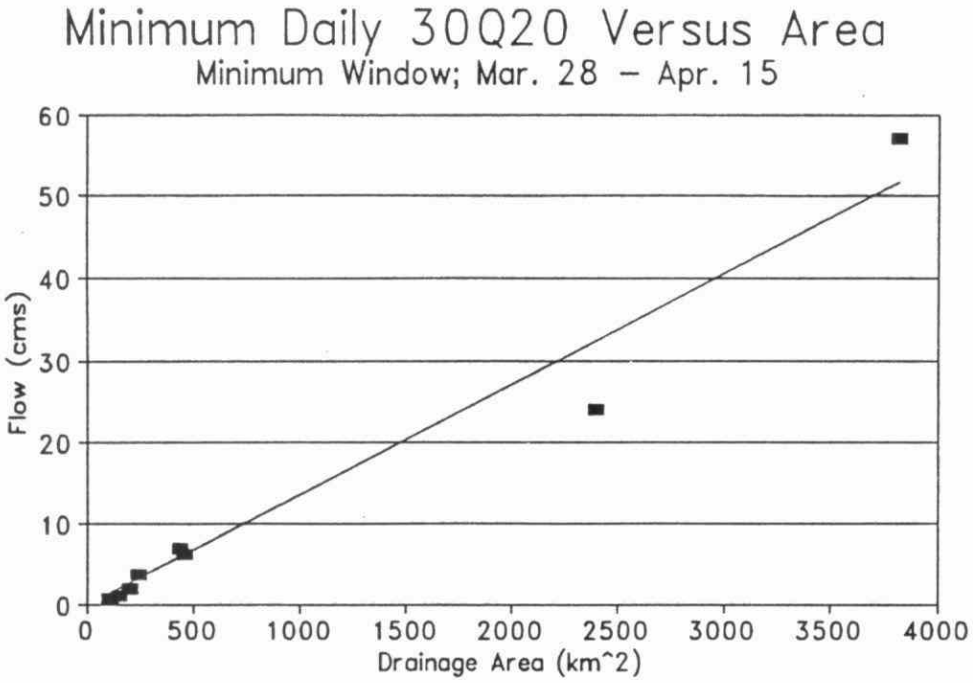

Figure 3-6 Spring 30Q20 Minimum Window Flow - Drainage Area Correlation

Maximum Window (Mar. 22 - May 13; 53 days):

$Q = 1.61 \times 10^{-3}(A_D)$ $r = 0.995$

Spring 30Q20

Minimum Window (Mar. 28 - Apr. 15; 19 days):

$Q = 1.35 \times 10^{-2}(A_D)$ $r = 0.980$

Average Window (Mar. 20 Apr. 20; 32 days):

$Q = 7.25 \times 10^{-3}(A_D)$ $r = 0.988$

Maximum Window (Mar. 16 - Apr. 24; 40 days):

$Q = 4.51 \times 10^{-3}(A_D)$ $r = 0.839$

Fall 30Q20

Minimum Window (Nov. 24 - Dec. 15; 22 days):

$$Q = 5.07 \times 10^{-4}(A_D) \qquad r = 0.921$$

Average Window (Nov. 16 - Dec. 17; 32 days):

$$Q = 4.58 \times 10^{-4}(A_D) \qquad r = 0.916$$

Maximum Window (Nov. 5 - Dec. 17; 43 days):

$$Q = 2.43 \times 10^{-4}(A_D) \qquad r = 0.867$$

Regressions for fall 7Q20 flows were not developed because there was too much scatter in the 7Q20 flow versus time plots. The discharge windows for the fall 7Q20 flows were estimated from the data as:

Fall 7Q20

Minimum Window: Nov. 27 - Dec. 14 (18 days); Average Window: Nov. 16 - Dec. 19 (34 days); Maximum Window: Nov. 8 - Dec. 28 (51 days)

The drainage areas at all stream confluences and existing or potential point wastewater discharge points were calculated according to techniques mentioned above. A plot of drainage area variation along the South Nation River is given in Appendix F. Also the distances between points was measured along streamflow paths. This information is displayed in the connectivity diagram shown in Fig. 3-7.

The flow data in Appendix F for Plantagenet and Casselman show that mean monthly flows at Casselman were higher than at Plantagenet for the months of February, September, October and December. Cumming Cockburn Ltd. (1989) also had similar aberrations in their results due to the raw data discrepancies for these two locations. The reason for these discrepancies was not resolved. February flows were not used for any of the window correlations developed above. But the discrepancies in the fall months contributed to lower correlations for the fall months. Even though some of the input data is suspect the equations are considered usable because of the high correlation coefficients.

The regression equations given above were used to calculate flows at all wastewater source locations for the spring 7Q20 and 30Q20 flows and for the fall 30Q20 flows. Fall 7Q20 flows at all stations were based on the fall 7Q20 flow for Plantagenet. Flows at Plantagenet are highest since it is the most downstream point in the watershed and the fall 7Q20 could be reliably gauged for this location. The fall 7Q20 flows at each of the other locations were estimated by taking the ratio of the drainage area for a location to the drainage area at Plantagenet and multiplying the fall 7Q20 flow at Plantagenet by this ratio. Spring and fall 7Q20 and 30Q20 flows at all point source locations are given in Tables 3-8 and 3-9, respectively.

These low flow estimates were used as a basis for the river modelling conducted in the present study, described in Chapter 6. In addition, the optimum discharge windows determined in the analysis could be used to develop specific

recommendations regarding discharge times at each of the municipal wastewater treatment plants. As shown from the analysis described above, the discharge time has a significant impact on the river volume available for dilution, and thus the quantity of effluent that can be discharged.

3.4 HYDRAULIC RELATIONS AND TRAVEL TIMES

To obtain the travel times between each node, channel geometry variation along the streams was required. Proctor & Redfern (1984) had a fairly detailed characterization in HEC2 format (US Army, 1979) of most of the South Nation River and all of the main branch of the Castor from Russell to its confluence with the South Nation River. There was no channel geometry information available for the other pertinent streams in the watershed.

All distances are reported as distance along the river upstream from confluence of the South Nation and Ottawa Rivers in Fig. 3-7. Channel geometry information for the South Nation River was unavailable for the reach between 41 and 57 km (this section lies between the Bear Brook confluence with the South Nation and Casselman) and the reach between approximately 100 and 130 km (between Chesterville and Spencerville). For these reaches, the average channel geometry, determined by the known reaches immediately before and after the unknown reach, was used. Otherwise the data available for the main branch of the Castor River and the South Nation River were fairly detailed. There were generally from 4 to 6 channel elevation measurements in the main channel at each location and the separation of data locations was generally significantly less than 1 km.

The 7Q20 and 30Q20 spring flows at each node were input along with the channel geometry information to the HEC2 model to obtain travel times between each node for each flow regime. These travel times, given in Table 3-10, were used in the models.

The approach used to obtain travel times in the East Castor, Scotch, Bear Brook and other small streams that receive effluents from the remote communities such as Maxville and Winchester is as follows. The flow relations used were

$$v = aQ^{0.4} \qquad \textbf{(3-1a)} \qquad\qquad d = bQ^{0.6} \qquad \textbf{(3-1b)}$$

where v is velocity, m/s
Q is volumetric flowrate, m^3/s
d is depth, m
a and b are constants

These relations are consistent with Manning's formula assuming a rectangular channel.

Flows were established according to the Q = f(drainage area) relationships given above. For any given river reach, the flow was incrementally increased between two points based on river length and additional contributing area.

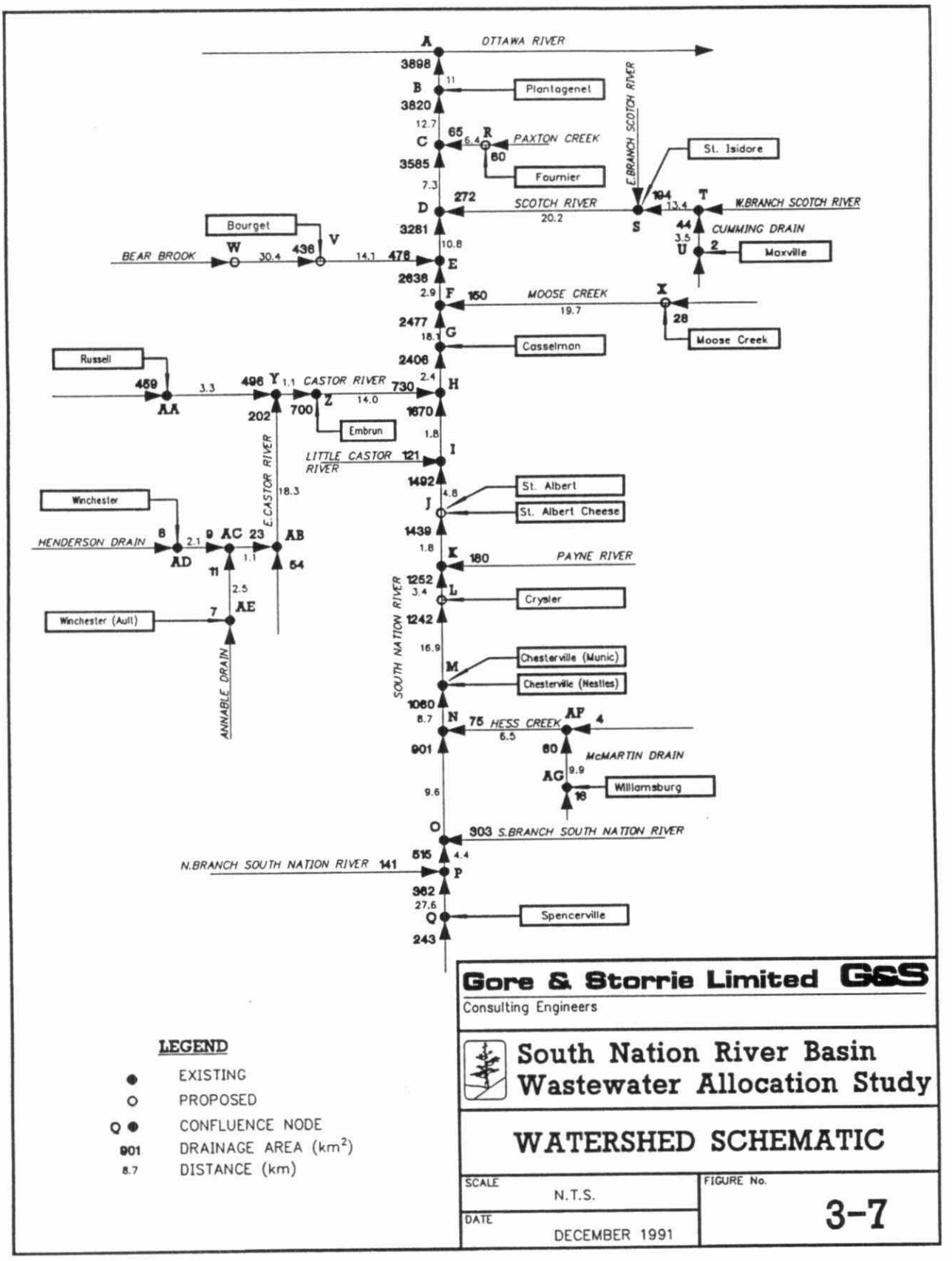

Figure 3-7 Connectivity Diagram for the Watershed

Table 3-8 Spring 7Q20 and 30Q20 Flowrates[1]

Municipality	Node	Drainage Area km²	7Q20 (m^3/s) Windows			30Q20 (m^3/s) Windows		
			Min. (15 d)	Ave. (33 d)	Max. (53 d)	Min. (19 d)	Ave. (32 d)	Max. (40 d)
Bear Brook	E	3112	24.24	10.18	5.01	42.01	22.56	14.04
Bourget	V	436	3.40	1.43	0.70	5.89	3.16	1.97
Casselman	G	2406	18.74	7.87	3.87	32.48	17.44	10.85
Castor River	H	2400	18.70	7.85	3.86	32.40	17.40	10.82
Chesterville	M	1060	8.26	3.47	1.71	14.31	7.69	4.78
Crysler	L	1242	9.68	4.06	2.00	16.77	9.00	5.60
Embrun	Z	700	5.45	2.29	1.13	9.45	5.08	3.16
Fournier	R	60	0.47	0.20	0.10	0.81	0.44	0.27
Hess Creek	N	976	7.60	3.19	1.57	13.18	7.08	4.40
Little Castor R.	I	1613	12.57	5.27	2.60	21.78	11.69	7.27
Maxville	U	2	0.02	0.01	0.00	0.03	0.01	0.01
Moose Creek	X	28	0.22	0.09	0.05	0.38	0.20	0.13
Paxton Creek	C	3650	28.43	11.94	5.88	49.28	26.46	16.46
Plantagenet	B	3820	29.76	12.49	6.15	51.57	27.70	17.23
Russell	AA	459	3.58	1.50	0.74	6.20	3.33	2.07
Scotch River	D	3553	27.68	11.62	5.72	47.97	25.76	16.02
Spencerville	Q	243	1.89	0.79	0.39	3.28	1.76	1.10
St. Albert	J	1439	11.21	4.71	2.32	19.43	10.43	6.49
St. Isidore	S	194	1.51	0.63	0.31	2.62	1.41	0.87
Williamsburg	AG	16	0.12	0.05	0.03	0.22	0.12	0.07
Winchester	AD	8	0.06	0.03	0.01	0.11	0.06	0.04
Ault	AE	7	0.05	0.02	0.01	0.09	0.05	0.03
Nestle	M	1060	8.26	3.47	1.71	14.31	7.69	4.78
St. Albert Cheese	J	1439	11.21	4.71	2.32	19.43	10.43	6.49

1. Estimated using the regression equations based on drainage area.

Table 3-9 Fall 7Q20[1] and 30Q20 flowrates[2]

Municipality	Node	Drainage Area km²	7Q20 (m^3/s) Windows Min. (18 d)	7Q20 (m^3/s) Windows Ave. (34 d)	7Q20 (m^3/s) Windows Max. (51 d)	30Q20 (m^3/s) Windows Min. (22 d)	30Q20 (m^3/s) Windows Ave. (32 d)	30Q20 (m^3/s) Windows Max. (43 d)
Bear Brook	E	3112	0.47	0.33	0.13	1.58	1.43	0.76
Bourget	V	436	0.07	0.05	0.02	0.22	0.20	0.11
Casselman	G	2406	0.37	0.25	0.10	1.22	1.10	0.58
Castor River	H	2400	0.36	0.25	0.10	1.22	1.10	0.58
Chesterville	M	1060	0.16	0.11	0.04	0.54	0.49	0.26
Crysler	L	1242	0.19	0.13	0.05	0.63	0.57	0.30
Embrun	Z	700	0.11	0.07	0.03	0.35	0.32	0.17
Fournier	R	60	0.01	0.01	0.00	0.03	0.03	0.01
Hess Creek	N	976	0.15	0.10	0.04	0.49	0.45	0.24
Little Castor R.	I	1613	0.24	0.17	0.07	0.82	0.74	0.39
Maxville	U	2	0.00	0.00	0.00	0.00	0.00	0.00
Moose Creek	X	28	0.00	0.00	0.00	0.01	0.01	0.01
Paxton Creek	C	3650	0.55	0.38	0.15	1.85	1.67	0.89
Plantagenet[3]	B	3820	0.58	0.40	0.16	1.94	1.75	0.93
Russell	AA	459	0.07	0.05	0.02	0.23	0.21	0.11
Scotch River	D	3553	0.54	0.37	0.15	1.80	1.63	0.86
Spencerville	Q	243	0.04	0.03	0.01	0.12	0.11	0.06
St. Albert	J	1439	0.22	0.15	0.06	0.73	0.66	0.35
St. Isidore	S	194	0.03	0.02	0.01	0.10	0.09	0.05
Williamsburg	AG	16	0.00	0.00	0.00	0.01	0.01	0.00
Winchester	AD	8	0.00	0.00	0.00	0.00	0.00	0.00
Ault	AE	7	0.00	0.00	0.00	0.00	0.00	0.00
Nestle	M	1060	0.16	0.11	0.04	0.54	0.49	0.26
St. Albert Cheese	J	1439	0.22	0.15	0.06	0.73	0.66	0.35

1. Estimated by taking the ratio of the drainage area at the location to the drainage area at Plantagenet and multiplying this ratio times the flow at Plantagenet.
2. Estimated using the regression equations based on drainage area.
3. The fall 7Q20 flows for Plantagenet are the actual 7Q20 flows determined by statistical analysis.

Table 3-10 Spring Travel Times[1] for All Nodes in the South Nation Watershed

	7Q20 Travel Time (d)			30Q20 Travel Time (d)		
	Window			Window		
Node	Min	Ave	Max	Min	Ave	Max
A	0	0	0	0	0	0
B	0.58	0.82	1.09	0.40	0.51	0.62
C	1.31	1.85	2.46	0.91	1.17	1.41
D	1.67	2.36	3.14	1.18	1.51	1.83
E	2.21	3.13	4.15	2.00	2.56	3.10
F	2.33	3.30	4.38	2.08	2.67	3.22
G	2.69	3.81	5.05	2.17	2.78	3.36
H	3.39	4.80	6.37	2.40	3.08	3.72
I	3.55	5.02	6.67	2.50	3.21	3.88
J	4.15	5.87	7.80	2.87	3.68	4.45
K	4.25	6.01	7.98	2.94	3.77	4.56
L	4.71	6.67	8.85	3.21	4.12	4.98
M	6.25	8.84	11.74	4.17	5.35	6.47
N	6.72	9.51	12.63	4.53	5.81	7.02
O	7.17	10.15	13.47	4.88	6.26	7.57
P	7.38	10.44	13.87	5.00	6.41	7.75
Q	8.70	12.31	16.35	6.01	7.71	9.32
R	2.82	4.02	5.33	2.14	2.75	3.32
S	4.66	6.60	8.75	4.58	4.59	5.56
T	7.36	10.43	13.81	5.75	7.37	8.93
U	8.87	12.58	16.65	6.97	8.93	10.82
V	3.80	5.39	7.14	3.28	4.21	5.09
X	6.84	9.71	12.85	5.71	7.33	8.86
Y	6.15	8.72	11.55	4.62	5.93	7.16
Z	6.01	8.52	11.29	4.51	5.78	6.99

Table 3-10 cont'd.

	7Q20 Travel Time (d)			30Q20 Travel Time (d)		
	Window			Window		
Node	Min	Ave	Max	Min	Ave	Max
AA	6.52	9.24	12.25	4.92	6.31	7.63
AB	9.53	13.53	17.91	7.34	9.42	11.39
AC	9.95	14.13	18.70	7.68	9.85	11.91
AD	11.11	15.78	20.88	8.61	11.05	13.33
AE	11.36	16.13	21.35	8.81	11.31	13.67
AF	8.25	11.68	15.49	5.76	7.38	8.93
AG	11.33	16.06	21.28	8.24	10.56	12.78

1. All travel times are travel time from the node to the Ottawa River.

Since there were no channel cross-section data, it appeared to be appropriate to apply average velocity and depth relationships for the small streams. The travel times were estimated by application of an average velocity determined by using Eq. 3-1a with an average value of the constant, a, determined from the HEC2 analysis of the main branch of the Castor River. The relationships applied for velocity and depth were as follows:

$$v = 0.062Q^{0.4} \quad \textbf{(3-2a)} \qquad d = 1.27Q^{0.6} \quad \textbf{(3-2b)}$$

The travel times determined for 7Q20 and 30Q20 flows at all nodes in Fig. 3-7 are given in Table 3-10. A plot of travel time along the South Nation River during the average spring window is given in Appendix F.

Particularly in the upstream reaches of streams, the waste flowrates are significant with respect to the streamflow. Travel times between nodes must be adjusted according to Eq. 3-1a when the flow in any reach is increased due to the addition of effluent from a point source. The equation used to adjust travel times between nodes when flow was augmented was

$$\Delta t' = \Delta t \left(\frac{Q_r}{Q_r + \Sigma Q_w} \right)^{0.4} \qquad \textbf{(3-3)}$$

where $\Delta t'$ is the corrected travel time
Δt is the travel time between nodes determined from Table 3-10 or 3-11
Q_r is the 7Q20 or 30Q20 flow in the reach

ΣQ_w includes the point source at the node and all point sources upstream of the node.

For the 7Q20 and 30Q20 fall flows, the flow regimes in many of the reaches change considerably and HEC2 was unable to compute the velocities. A more detailed channel characterization would be required to use HEC2 to calculate the velocities under these flow conditions. Pool and ripple flows exist at the 7Q20 and 30Q20 conditions.

With the information available, the only approach available to estimate 7Q20 and 30Q20 travel times was to use Eq. 3-2a. The travel time in a reach for the 7Q20 fall flow was obtained by multiplying the travel time obtained for the maximum spring 7Q20 flow by the ratio of the spring maximum window 7Q20 flow divided by the fall 7Q20 flow raised to the 0.4 power. The maximum window 7Q20 flows were the smallest flows able to be run through HEC2 and obtain credible results. Therefore these flows and travel times provided the best basis for estimating fall travel times.

The fall 30Q20 travel times were obtained in the same manner as the 7Q20 travel times. Again the spring maximum window flow and travel times were used as the basis.

Equations 3-2a and 3-2b were used to estimate the travel times in the small tributaries for both fall 7Q20 and 30Q20 flows.

The approach used for the fall travel times is crude and results should be interpreted with caution. The travel times for all fall flows are given in Table 3-11.

Table 3-11 Fall Travel Times[1] for All Nodes in the South Nation Watershed

	7Q20 Travel Time[2] (d)			30Q20 Travel Time (d)		
	Window			Window		
Node	Min	Ave	Max	Min	Ave	Max
A	0	0	0	0	0	0
B	2.80	3.25	4.69	1.73	1.80	2.32
C	6.33	7.34	10.59	3.91	4.07	5.24
D	8.07	9.36	13.51	4.98	5.19	6.68
E	10.68	12.39	17.87	6.59	6.87	8.85
F	11.26	13.06	18.84	6.95	7.24	9.33
G	13.00	15.08	21.75	8.02	8.36	10.77
H	16.38	19.00	27.42	10.11	10.53	13.57
I	17.15	19.90	28.71	10.59	11.03	14.21
J	20.05	23.26	33.56	12.38	12.89	16.61
K	20.53	23.82	34.37	12.68	13.20	17.01
L	22.76	26.40	38.09	14.05	14.63	18.85
M	30.20	35.03	50.54	18.64	19.42	25.02
N	32.47	37.67	54.35	20.04	20.88	26.90
O	34.64	40.19	57.98	21.39	22.27	28.70
P	35.66	41.37	59.68	22.01	22.93	29.54
Q	42.03	48.77	70.36	25.95	27.03	34.83
R	13.72	15.94	22.95	8.47	8.83	11.36
S	22.51	26.16	37.65	13.90	14.50	18.64
T	35.56	41.33	59.43	21.95	22.90	29.42
U	42.85	49.84	71.66	26.46	27.62	35.47
V	18.36	21.34	30.74	11.33	11.83	15.22
X	33.05	38.45	55.30	20.40	21.31	27.37
Y	29.71	34.53	49.71	18.34	19.57	25.15

Table 3-11 cont'd.

	7Q20 Travel Time[2] (d)			30Q20 Travel Time (d)		
	Window			Window		
Node	Min	Ave	Max	Min	Ave	Max
Z	29.04	33.74	48.59	17.93	19.13	24.59
AA	31.50	36.59	52.72	19.45	20.71	26.64
AB	46.04	53.58	77.09	28.43	30.12	38.70
AC	48.07	55.96	80.49	29.68	31.44	40.38
AD	53.68	62.50	89.87	33.14	35.06	45.03
AE	54.88	63.88	91.89	33.88	35.83	46.03
AF	39.86	46.26	66.66	24.61	25.64	32.99
AG	54.74	63.61	91.58	33.79	35.25	45.33

1. All travel times are from the node to the Ottawa River.
2. Fall 7Q20 travel times were based on Eq. 6-2a.

SECTION 4

RIVER WATER QUALITY

RIVER WATER QUALITY

4.1 INTRODUCTION

River water quality fluctuates with natural and manmade inputs. This phase of the study was focused on assessing the background stream quality in the watershed.

4.2 RIVER QUALITY ANALYSIS

An analysis of river water quality parameters for all stations that have been monitored by the MOE (MOE, 1991b) was conducted. The data set generally spans the period 1966 through 1991 for the major sampling stations given in Table 4-1. Data from other stations were available in this data set but they were only monitored for periods of time from 2 to 5 years before 1980. Because of the short period of time for which these stations were in existence there are only a small number of measurements available for these stations. Furthermore the sampling periods were earlier than 10 years ago. Population growth and wastewater treatment conditions have changed significantly during the past 10 years as discussed in Chapter 2. For these reasons these stations were generally eliminated from consideration.

Two water treatment plants have been taking daily measurements of the river's pH and temperature. Data from the Plantagenet and Casselman water treatment plants is available for the past 11 and 5 years respectively. It has been requested by the South Nation River Conservation Authority that data from these two sources be used in the optimization model.

Table 4-1 River Water Quality Sampling Stations

Station	MOE Station ID	Sampling Period
Casselman (South Nation)	18-2070-100-02	1966-91
Chesterville (South Nation)	18-2070-110-02	1966-91
Plantagenet (South Nation)	18-2070-020-02	1966-91
Russell[1] (Castor)	18-2070-140-02	1980-91
Russell[2] (Castor)	18-2070-145-02	1980-91
St. Isidore (Scotch River)	18-2070-040-02	1969-91
St. Isidore[3] (Scotch River, upstream)	18-2070-060-02	1968-91

1. Designated Russell (Con. 5) in other tables.
2. Designated Russell (Con. 3) in other tables.
3. Designated St. Isidore (upper) in other tables.

The stations at Russell, although on different branches of the Castor River, are fairly close and sometimes only one of the stations is reported as representative of water quality in the Castor system. Likewise, the two stations at St. Isidore are within 5 km

of each other on the same branch of the Scotch River and at times only one of them is reported. The notes below the table give the naming conventions for these stations. The quality monitoring stations are located at or near the stream gauging stations shown in Fig. 3-1.

Statistics on various water quality parameters have been compiled for each of the stations in Table 4-1 on a monthly and spring and fall basis. Spring is defined as the months of March, April and May; fall is defined as the months of October, November and December. A complete listing of the statistics is given in Appendix F. Statistics for the Plantagenet and Casselman water treatment plants is given in Appendix H.

Data for the parameters of interest (pH, BOD_5, temperature, DO, nitrogen compounds and TP) were also plotted for the years 1986-1991 at each of the stations in Table 4-1 to identify any trends. Plots for the Plantagenet station, which are characteristic of the other plots, are shown in Figs. 4-1a - 4-1d. It should be noted that these plots do not include water treatment plant data. Similar plots of the river water quality parameters for the Casselman and Chesterville stations are given in Appendix F.

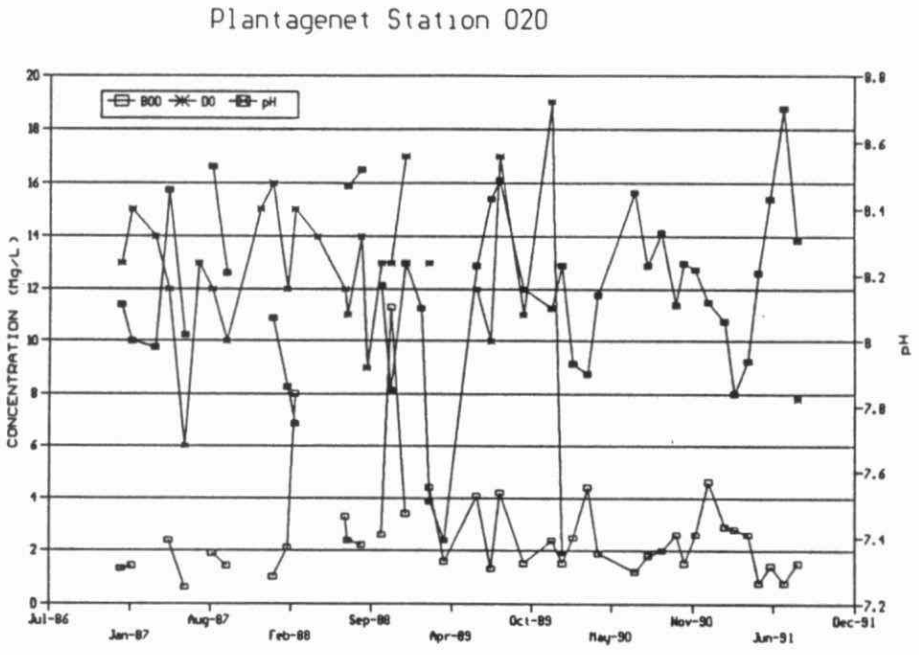

Figure 4-1a DO, pH and BOD_5 Variation at Plantagenet for 1987-90

Besides temperature and flow which exhibit characteristic seasonal variation, the only other parameter which showed a cyclical pattern is nitrate. Nitrate tends to peak around the months of November to December. These late fall peaks in nitrate should

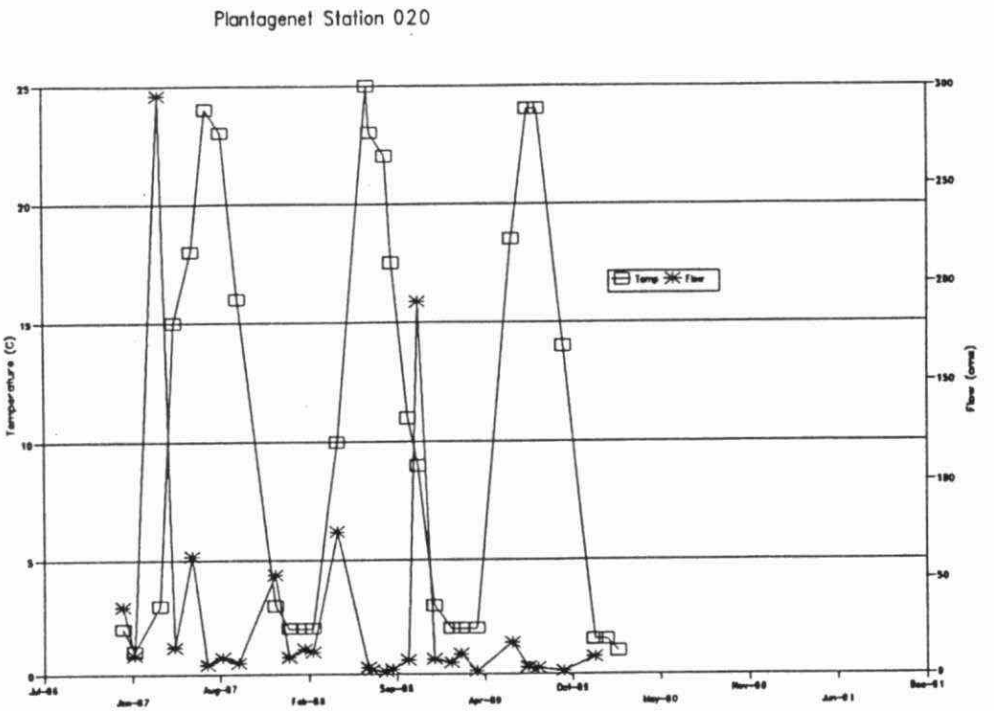

Figure 4-1b Flow and Temperature Variation at Plantagenet for 1987-90

somewhat be a result of the discharge of highly nitrified effluent from some of the lagoons.The data on the river are not generally suitable for calibration of a river quality model. There were some months where two grab samples were taken and analyzed but, in general, there is only one sample at most for a month and there are a considerable number of months where no measurement of a given substance was made. The only parameters for which a significant amount of data was available are pH and temperature. These parameters, however, were measured at two locations only (the Plantagenet and Casselman water treatment plants).

Data on the river system have been gathered at most stations for a period of approximately 25 years. Data from years before 1980 are not as relevant to the analysis for many parameters because waste sources discharging into the river have changed. Many of the communities did not have any wastewater conveyance or treatment facilities before 1980. Septic tank systems in varying states of functionality or direct discharge to a receiving stream would have been the only alternatives available for servicing residents of communities before sewerage was installed.

River water quality is also highly affected by agricultural practices. More recent quality data would more accurately reflect the results of current agricultural practices in the watershed.

4.2.1 River Water Quality PWQO Violations

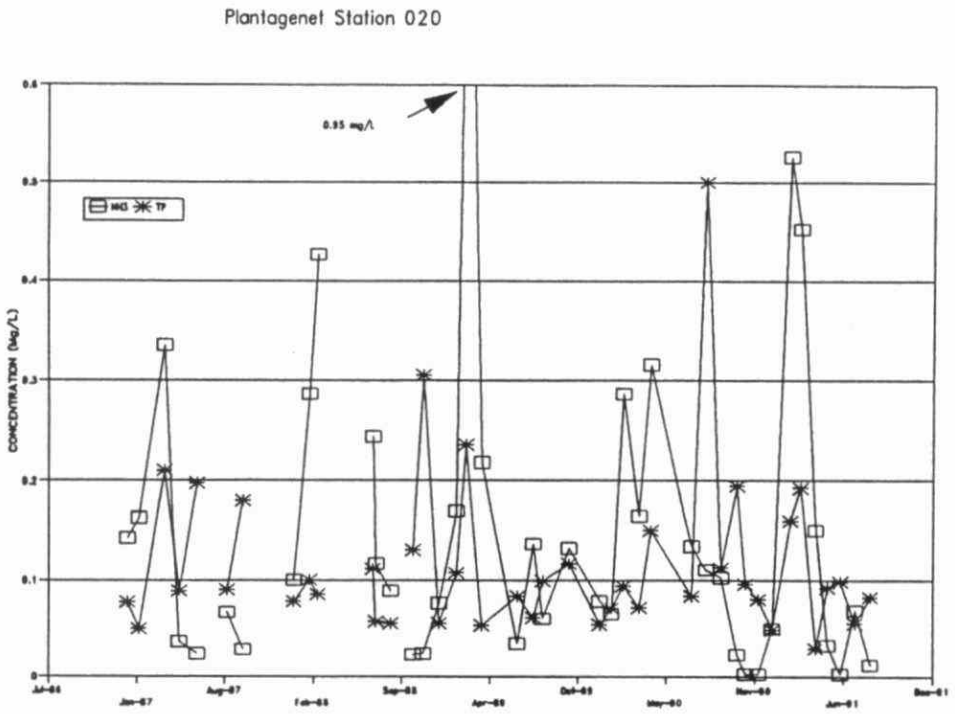

Figure 4-1c Ammonia and Phosphorus Variation at Plantagenet for 1987-90

The river water quality data at the stations given in Table 4-1 and all of the other stations for which sampling was only conducted over a few years have been scanned for violations of Provincial Water Quality Objectives (PWQO; MOE, 1984) standards. The relevant criteria and parameters are given in Table 4-2. There is no PWQO for BOD_5 and the criterion used for it is discussed below. For DO, the rivers were considered as warm water streams. The temperature of the stream was not always reported when DO measurements were taken. In these circumstances the DO criterion was set at the minimum value of 4.0 mg/L.

The equation used to determine the concentration ratio of un-ionized to ionized ammonia as a function of temperature and pH was

$$\frac{[NH_3]}{[NH_4^+]} = 10^{[pH - 10.08 + 0.0427T^{(0.918)}]} \qquad (4\text{-}1)$$

where T is in °C

This equation was determined from a regression analysis of the MOE (1984b) ammonia dissociation data. Using this equation the fraction of un-ionized ammonia could be calculated from the NH_{3T}-N concentrations. As for DO, there were a number of instances where the NH_{3T}-N concentration was reported at a station but the temperature and/or pH were not reported. The above equation could not be used in these instances and only potential violations could be assessed. In this case the

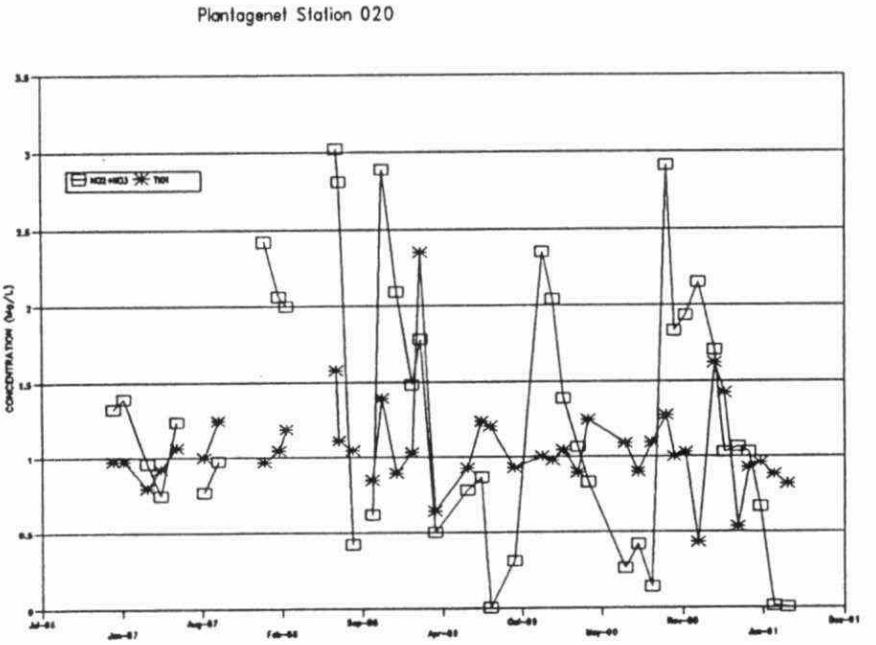

Figure 4-1d Nitrite and Nitrate Variation at Plantagenet for 1987-90

percent of un-ionized ammonia was assumed to be 2.16% based on the mean pH and temperatures for the spring and fall at all stations. From the spring and fall data in Appendix F, the mean temperature was 8.01°C and the mean pH was 8.13.

The PWQO do not specify a value for BOD_5 concentration, the issue is only indirectly addressed through DO specifications. The TOR specify that at the periphery of the mixing zone BOD_5 should be less than 6 mg/L. There are a number of indices for classifying stream quality. Prati's classification system (Ott, 1978) for surface water quality specifies that a BOD_5 concentration less than 3.0 mg/L is acceptable and a stream is "slightly polluted" at a BOD_5 concentration of 6.0 mg/L or greater.

The number of violations of the criteria noted in Table 4-2 and discussed above for all river water quality data in the watershed, are given in Table 4-3. The number of DO violations will be greater than the number reported in Table 4-3 because at lower temperatures the DO concentration criterion rises above 4 mg/L which was the concentration used when temperature data were unavailable. The number of violations for all years is reported and in addition the number of violations occurring during the last 12 years is also given.

Phosphorus is the most frequently monitored parameter with 1300 measurements over the complete monitoring period. Some parameters were not monitored as frequently as others. Phosphorus concentrations are almost always higher than the objective concentration.

Table 4-2 PWQO and BOD_5 Criteria

Parameter	DO	BOD_5	Un-ionized NH_3	TP
Criteria	47%[1] Saturation	6 mg/L	0.02[2] mg/L	0.03 mg/L

1. When the temperature was unknown, a value of 4 mg/L was used.
2. When the temperature or pH were not reported, free ammonia was evaluated as 2.16% of NH_{3T} based on the mean pH and temperature for the spring and fall periods.

The most interesting point to be made from Table 4-3 is that the frequency of violations has substantially decreased during the last 12 years for un-ionized ammonia, BOD and DO. The installation of treatment facilities which have replaced malfunctioning septic tank disposal systems and other direct discharge sources is undoubtedly responsible for a portion of the improvement. Changes in agricultural practice are also another factor of importance. Data and information available for this study were unable to quantify the extent of any of these.

Table 4-3 Number of Violations of PWQO and BOD_5 Criteria

	Un-ionized NH_3				
Parameter	Known	Potential	DO[2]	BOD_5	TP
Number/Total (all years)	32/1190 (2.7%)	62[1]/1190 (5.2%)	207/1270 (16%)	99/1115 (8.9%)	1248/1300 (96%)
Number/Total (1980-1991)	11/438 (2.5%)	1[1]/438 (0.2%)	28/543 (4.7%)	0/325 (0%)	587/595 (99%)

1. The potential violations do not include the known violations.
2. The number of DO violations will be larger than this number for reasons explained in the text.

This conclusion does not apply to phosphorus which has exhibited further deterioration of an already extremely high frequency of violations. It will be shown below that the phosphorus load contributed by municipal effluents is a small fraction of the phosphorus load in the river.

4.3 POLLUTANT LOADS IN THE RIVERS

To calculate pollutant loads in the rivers, missing river data were handled in a manner somewhat similar to missing effluent data discussed in Secs. 2.2 and 2.3. As noted above there was generally only one measurement of any parameter during a month and often times there was no measurement. When river concentration data were unavailable for a month, the average concentration based on existing data for a year was used as the concentration. Since there was no evident trends in concentrations of any parameters except nitrates (which appeared to peak in Nov.-Dec.), this approach

is more reasonable than using the historical average concentration for a month. River concentrations change as treatment facilities are added or improved and are somewhat influenced by the flow and rainfall that occurred. The changes occurring in treatment over the past few years favour considering each year as a unit.

When flow data were missing at a station for a few days (up to 14 days) only, the missing data were replaced by taking the average flow calculated from the last day before and the first day after the missing data set. When flow data were missing for a longer period of time, the flow data were replaced with values obtained by multiplying the flow measured at Plantagenet on the missing days by the ratio of the flow at the flow station to the flow at Plantagenet. The flow data set at Plantagenet was complete. The ratio of the flow at the station to the flow at the Plantagenet station was determined from flow measurements at both stations immediately before and after the missing data set.

The complete data set including concentrations with a breakdown of nitrogen data into each of the nitrogen forms (nitrite, nitrate, ammonia and TKN) and all monthly loadings for the river stations is given in Appendix E. In addition, background data and calculated monthly pollutant loads being discharged by all municipal and industrial are given in Appendix E. Summary tables comparing point source loads on a monthly and annual basis for the years 1986-90 are given in this appendix.

4.4 BACKGROUND RIVER CONCENTRATIONS AND NONPOINT SOURCE LOADS

The calculated annual loadings during the years 1986-1990, extracted from tables in Appendix E, for all parameters for all municipal and industrial sources and the river stations are given in Table 4-4. Not all of the municipal lagoons were operating during the earlier years (refer to Appendix E).

Total nitrogen contributions to the river from all wastewater facilities ranged from 0.4 to 1.0% of the total nitrogen load in the South Nation River (measured at Plantagenet) over the years 1986-90. TP contributions to the river from all wastewater sources ranged from 3.0 to 6.7% of the TP load in the river at Plantagenet for the same time period. These values are in agreement with a previous study which found the N and P point source contributions to the river load to be near 1 and 4%, respectively (MacLaren, 1982). There has not been any significant change in the relative contributions of the wastewater sources during the past 10 years for these nutrients.

It is apparent that the total contribution of these nutrients from all wastewater sources is minimal compared to the nonpoint and other sources. Some of the background concentrations of N and P in the river are due to poorly functioning onsite wastewater disposal systems at households that are not connected to a sewerage system with a treatment facility. Rectifying this situation at Crysler, Moose Creek, Fournier and St. Albert may have a significant impact on the background concentrations of these nutrients but further study will be required to fully assess the impact.

The nonpoint sources of nutrients which are largely agricultural must be reduced significantly before augmentation of sewage treatment with nutrient removal, in

particular, alum treatment for phosphorus, will have any significant effects on the health of the river from a nutrient standpoint. From data reported in Chapter 2, TP levels were typically being reduced from 10 to 2 mg/L or less without alum treatment. Adding alum to remove P down to 1 mg/L or less will not result in any significant impact on the river.

The ratio of ammonia loads from point sources to the load in the South Nation River at Plantagenet varied from 3.6 to 5.2% using the data in Table 4-4. Ammonia is chemically and biochemically transformed in the river. Comparing the point source contributions of NH_{3T} to the load in the river at Plantagenet (the furthest downstream station), therefore will overestimate the fraction of the river ammonia load that originates from point sources.

From the data in Table 4-4, the BOD_5 loadings from all wastewater sources are less than 1% of the BOD_5 loads in the river evaluated at Plantagenet on an annual basis. Comparing monthly BOD_5 loadings during the months when lagoons were discharging (Appendix E) the BOD_5 loads from all wastewater sources are generally less than 5% of the total load in the river at Plantagenet.

Table 4-4 Annual Loadings for Wastewater Sources and River Stations

	Loading (kg/yr)				
	N	NH_{3T}-N	P	BOD_5	SS
1986					
All Wastewater Sources	18,070	5,642	9,326	28,064	54,334
River Stations					
Plantagenet	3,447,558	126,168	151,333	3,222,343	-
Casselman	2,336,566	45,801	110,624	1,310,176	-
Russell	407,777	4,577	21,730	196,633	-
St. Isidore	173,829	3,820	13,138	73,391	-
1987					
All Wastewater Sources	19,223	7,894	5,023	19,229	40,906
River Stations					
Plantagenet	2,288,895	186,211	165,526	1,931,744	-
Casselman	1,637,326	108,394	61,847	1,269,183	-
Russell	335,908	16,712	12,274	340,071	-
St. Isidore	206,670	10,026	13,092	259,492	-

Table 4-4 Cont'd.

	Loading (kg/yr)				
	N	NH_{3T}-N	P	BOD_5	SS
1988					
All Wastewater Sources	21,180	8,787	5,049	24,542	47,977
River Stations					
Plantagenet	3,295,558	169,851	133,130	5,556,705	-
Casselman	1,368,644	68,167	38,589	708,591	-
Russell	178,358	8,392	10,144	148,365	-
St. Isidore	104,360	6,843	8,871	94,980	-
1989					
All Wastewater Sources	18,718	7,258	3,378	17,294	72,081
River Stations					
Plantagenet	1,812,390	171,329	87,455	2,449,357	-
Casselman	1,338,154	35,458	50,700	933,519	-
Russell	79,011	1,278	4,965	108,977	-
St. Isidore	49,731	6,010	5,840	89,529	-
1990					
All Wastewater Sources	18,762	9,784	1,283	20,295	44,280
River Stations					
Plantagenet	4,348,243	268,476	192,664	4,589,201	-
Casselman	2,465,929	117,672	90,554	1,816,500	-
Russell	308,850	6,398	8,560	215,882	-
St. Isidore	123,277	3,013	5,972	92,420	-

The background concentrations of NH_{3T} and BOD are required for the DO modelling. It has been shown that BOD and NH_3 river PWQO violations have significantly decreased during the last 12 years (Table 4-3) which also indicates that background concentrations of these parameters have changed significantly. Therefore river data for 1980-91 only at the 7 stations in Table 4-1 were statistically evaluated on a monthly basis (Appendix F). The listing shows that for many months the total number of samples taken over the 11 year period was too small too extract meaningful statistics. Therefore the data were combined on a seasonal basis as before: spring is defined as the months of March, April and May; and fall is defined as the months of

October, November and December. The spring and fall statistics for the seven stations are given in Tables 4-5 and 4-6, respectively.

Monthly loads of ammonia from point sources reached high values of 52 and 66% in the spring and fall, respectively (see Appendix E). But for the 30 spring and fall monthly loads calculated, there were only 10 during which the total point source load was more than 10% of the river load at Plantagenet. Subtracting the point source loads from the river loads would only result in small changes in the 75 percentile ammonia values in Tables 4-5 and 4-6. Therefore these changes were ignored.

The spring data set for Plantagenet exhibited unusually high BOD_5 values compared to the other locations. The sample of 14.3 mg/L was considered to be an outlier that unduly influenced the statistics from the small number of samples. The statistics for the modified set are also included in Table 4-5. More representative results are obtained with the modified statistics.

Table 4-5 Spring Quality Statistics for 1980-91 Data

						Percentiles		
Station	Parameter[1]	No.	Max.	Mean	Min.	75%	50%	25%
Plantagenet	BOD_5	10	14.30	4.03	0.80	4.40	2.50	1.60
	BOD_5[2]	9	8.00	2.89	0.80	2.75	2.15	1.53
	NH_3-N	24	0.45	0.14	0.004	0.23	0.10	0.03
St. Isidore	BOD_5	21	7.10	2.05	0.40	1.93	1.58	1.20
	NH_3	22	0.42	0.12	0.006	0.15	0.07	0.02
St. Isidore (upper)	BOD_5	8	2.50	1.64	0.60	2.30	1.75	0.95
	NH_3-N	9	0.13	0.06	0.012	0.08	0.07	0.03
Casselman	BOD_5	8	1.70	1.11	0.60	1.60	1.05	0.65
	NH_3-N	11	0.28	0.11	0.010	0.12	0.10	0.07
Chesterville	BOD_5	7	3.30	1.64	0.50	2.30	1.40	0.90
	NH_3-N	8	0.14	0.05	0.004	0.06	0.04	0.02
Russell (Con. 5)	BOD_5	4	1.50	1.03	0.20	1.35	1.20	0.70
	NH_3-N	4	0.09	0.07	0.036	0.09	0.06	0.04
Russell (Con. 3)	BOD_5	21	2.80	1.19	0.22	1.40	1.07	0.74
	NH_3-N	22	0.29	0.05	0.004	0.05	0.03	0.02

1. Parameter concentrations are expressed in mg/L.
2. The value of 14.3 mg/L was eliminated from this data set.

Table 4-6 Fall Water Quality Statistics for 1980-91 Data

						Percentiles		
Station	Parameter[1]	No.	Max.	Mean	Min.	75%	50%	25%
Plantagenet	BOD_5	11	11.3	2.79	0.80	2.60	2.40	1.00
	NH_3-N	26	0.44	0.08	0.002	0.12	0.04	0.02
St. Isidore	BOD_5	20	5.60	2.06	0.57	3.20	1.62	1.05
	NH_3-N	23	0.38	0.09	0.002	0.13	0.07	0.03
St. Isidore (upper)	BOD_5	6	2.70	1.85	0.70	2.40	2.15	1.00
	NH_3-N	10	0.04	0.03	0.01	0.04	0.03	0.02
Casselman	BOD_5	9	2.80	1.43	0.40	1.80	1.30	1.00
	NH_3-N	14	0.14	0.06	0.01	0.09	0.07	0.03
Chesterville	BOD_5	8	3.00	1.64	0.90	2.20	1.30	1.10
	NH_3-N	11	0.13	0.04	0.004	0.06	0.03	0.02
Russell (Con. 5)	BOD_5	10	5.10	1.49	0.30	1.80	1.05	0.70
	NH_3-N	10	0.05	0.03	0.002	0.03	0.02	0.02
Russell (Con. 3)	BOD_5	20	3.60	1.31	0.40	1.55	1.05	0.69
	NH_3-N	23	0.04	0.02	0.002	0.03	0.02	0.01

1. Parameter concentrations are expressed in mg/L.

From examination of the data in Tables 4-5 and 4-6 the following values are reasonable and conservative values for BOD_5 and NH_{3T} for both spring and fall:

BOD_5: 2.3 mg/L
NH_{3T}-N: 0.15 mg/L
(NH_{3T}: 0.18 mg/L)

The above values are near the highest 75 percentile values recorded at any river station and will be used as uniform input background concentrations into the rivers. According to Prati's (Ott, 1978) classification system (Sec. 4.2.1), a BOD_5 of less than 3.0 mg/L is acceptable and from the data, the South Nation River system normally has concentrations below this value.

The background concentrations reflect the net result of a number of phenomena. Nonpoint sources (groundwater and surface water), benthic activity and transformations, direct and indirect wastewater sources such as household and farm systems and instream chemical and biochemical activity all influence the background concentrations.

Both ammonia and BOD_5 concentrations exhibit a rise at Plantagenet compared to the other stations. Part of the reason for this rise is due to the cumulative effects of all

upstream sources, particularly addition of effluent point sources (although their effect is not too large) during the spring and fall discharge periods. Therefore it is more reasonable to use background concentrations at upstream locations.

From the data in Table 4-4 and the tables in Appendix E it is not necessary to make any significant adjustments in the background concentrations due to the contributions of the known municipal and industrial point sources because their loads in general comprise a very small component of the total stream loads.

4.5 NONPOINT SOURCE POLLUTION

4.5.1 NPS Pollution In The South Nation River Watershed

The South Nation River Watershed covers an area of approximately 390,000 ha, most of agricultural. Within this overall land use mosaic are fifteen municipalities ranging in size from 230 to 3500 persons. Loadings from municipal effluents have been evaluate in the above sub-sections of this chapter. It was observed that the number of violations of PWQO Criteria for un-ionized ammonia, BOD, and DO has been substantially decreased since the installation of municipal treatment facilities. It is speculated that these facilities directly reduced waste loadings from dysfunctional septic tank systems. However, rural septic treatment facilities were not upgraded.

In relative terms Point Source BOD loadings were less then 1 percent of total loads in the river on an annual basis, and less then 5 percent on a monthly basis when the lagoons are discharging. Similarly, the ratio of ammonia loads from point sources to total ammonia loads in the river at Plantagenet varied form 3.6 to 5.2 percent. Total Nitrogen contributions from municipal facilities ranged from 0.4 to 1.0 percent of the total Nitrogen load in the river. Phosphorus concentrations almost always exceed PWQO Criteria, however, Total Phosphorus loadings from municipal effluents comprise only 3.0 to 6.7 percent of the total TP load in the river at Plantagenet. These observations support previous findings (MacLaren, 1982) which show that loadings from municipal point source contamination constitutes a small portion of the total BOD, TP, and nitrogen loading to the South Nation River. This implies that other potential sources of contaminants to the South Nation River system, such as stormwater outfalls, landfill sites, commercial effluent, septic tank systems, agricultural practices and runoff from agricultural areas may be the principal anthropogenic sources of contamination.

Observations from the Rural Beaches Strategy Program studies conducted by the Halton Region Conservation Authority (HRCA), the Upper UTRCA and other Authorities as summarized above note that grey wastes and waste loadings from dysfunctional septic tank systems may contribute significant amounts of ammonia, bacteria and phosphorus; livestock access to streams and drains may contribute to bacteria pollution, and bank erosion; manure spills and runoff from manure piles may also contribute to bacterial contamination; milkhouse washwater may contribute to TP loadings, and bacteria; and, soil erosion is the single highest contributor of phosphorus. Given the extensive agricultural and animal husbandry related activities within the South Nation River watershed control of possible NPS pollution from these

sources may constitute the single most cost effective measure for the remediation of river water quality.

4.5.2 *NPS Modelling And Abatement Programs In Ontario*

Rowney and MacRae (1989) conducted a telephone survey of the Conservation Authorities in the Eastern Ontario region, the Provincial Ministries of the Environment, Agriculture and Food, and selected consulting firms active in nonpoint source (NPS) pollution investigations. The survey concluded that NPS contamination of receiving waters is recognised as a real and significant problem in Eastern Ontario and that this problem is growing in importance in both perceived and real terms. There were, however, no comprehensive and co-ordinated political infrastructures, or funding and implementation mechanisms in place to address the NPS issue. There were, however, a number of small demonstration studies and abatement programs which had been or were in the process of being implemented. The general experience gathered through programs with cottage owners and farm associations in the form of low cost self help programs has been very encouraging. Field services programs such as Ontario Ministry of Agriculture and Food's (OMAF's) "Land Stewardship" program and the "Soil and Water Environmental Enhancement Program" (SWEEP), have also been successful.

One study of particular interest was the conducted under the auspices of "A Clean Up Rural Beaches Plan" (CURB; developed by the Upper Thames River Conservation Authority [UTRCA] and funded by the MOE; Hayman, 1988). This study involved the development of an NPS abatement program for the Fanshawe, Pittock and Wildwood reservoirs using the CURB model for the estimation of NPS loadings and the evaluation of remedial plans. The soil types, land use activities and land use management practices in the study watersheds are similar to those found in the South Nation River watershed. Some of the principal findings from this study were:

- erosion related phosphorus sources were approximately equal to the annual input from all other identified sources (milkhouse wastes, septic effluent, livestock access, manure storage runoff, overspreading, winterspreading and urban);

- however, milkhouse waste treatment was considered to be the most cost effective method for the control of phosphorus loadings to the receiver; and,

- improvements to rural resident septic tank systems was identified as the second most cost-effective measure for the control of both fecal coliform and phosphorus contamination.

Palmateer (1991) has found similar results. He reported that malfunctioning septic tanks on farms are usually re-routed to adjacent field tile drains rather than having repairs made to the septic tank system. In addition, household greywater is also routinely routed into field tile or open drains to lengthen the life of septic tank systems. Milkhouse wastes and liquid barnyard wastes are often transferred to the field tile system due to lack of treatment.

SNRCA (1990) studied the Payne River in the South Nation watershed for NPS pollution. The study found that both excessive phosphorus pollution and coliform densities in this river were due primarily to unlimited cattle access to the river. Elevated levels of turbidity, BOD and nitrogen were associated with fecal contamination. Septic tank systems located near the Payne River were ruled out as a major source of contamination because of their low density.

All of these sources and liquid manure spreading on tiled field can result in extremely high concentrations of carbon, nutrients and bacteria going to the receiving stream when the tiles are running due to runoff or high groundwater table.

Hayman (1992) noted that the UTRCA are currently involved in a continuation of the Kintore demonstration control study, but at a larger scale, and a number of small sub-watershed studies as part of the CURB program. The Kintore studies have shown that conservation tillage can produce a reduction of 20-30% in sediment and 30% in phosphorus loadings. These reductions were supported through model studies using the universal soil loss equation (USLE) method by varying crop management practices (C factor).

Results from the CURB program studies are presented in annual reports published by the UTRCA. In general these studies confirm the results obtained through the CURB model investigations reported by Hayman (1988) and summarized above. An adaptation of the CURB model is currently being developed by Gore & Storrie Limited for integration into the QUALHYMO model as a means of predicting NPS loadings and the impact of remedial plans.

The OMAF field programs (Land Stewardship and SWEEP) were in their final stages at the time of the original survey by Rowney and MacRae (1989). However, a number of new programs, i.e., Land Stewardship II and two federal programs from Agriculture Canada have been instituted to replace them. The Land Stewardship II program is a provincial effort with a 3 year mandate. A total of 27.6 million dollars is available to farmers through grants which are allocated based on row crop acre by county. This means that the bulk of the funds will be focused on Southwestern Ontario.

The program is divided into two specific components: conservation practices such as conservation tillage; and, environmental farming. The later component deals with issues of milkhouse waste treatment, proper storage facilities for pesticides, etc. The program, which is now in its second year, has been very well received but the majority of funds have already been committed.

The federal programs consist of a Soil Conservation Program and an "Ontario Farm Groundwater Survey". The former program budgeted 8-9 million dollars for Ontario farmers (up to $10,000 per applicant) to establish permeant cover for the retirement of fragile lands. Monies are allocated on a row crop acre basis by county. This program is also entering its final stage and funds are largely exhausted. It is hoped that Agriculture Canada will continue or follow up with a similar program.

Other NPS abatement programs are currently under consideration at OMAF to replace current programs which are now being concluded. More specific information will be available through OMAF by mid-summer 1992.

4.5.3 *Evaluating Remediation Measures*

Several projects have been initiated in Ontario to remediate NPS pollution from agricultural areas. These projects include: fencing alternatives for restricting livestock access to streams; development of farm plans (crop rotation, contouring, strip cropping, conservation tillage, terracing, tile drainage, tree windbreaks, and the retirement of highly erodible slopes and flood plain lands) to control soil erosion off fields; techniques for increasing the life expectancy of milkhouse wastewater treatment systems; improvements to septic waste treatment systems and manure storage facilities. On a farm unit basis these studies have demonstrated that remediation measures can significantly reduce NPS loadings. Briggs (1990) observed a significant decrease in TP concentrations with the implementation of improved milkhouse washwater and manure storage facilities including the construction of retaining walls around the feedlot. The geometric mean total phosphorus concentration at another farm outlet declined by a factor to 10 from 4.15 mg/l to 0.44 mg/l following disconnection and treatment of waste discharges.

The success of soil conservation practices on water quality, particularly suspended solids and phosphorus reduction, in the Avon River demonstration study has proven to be more difficult to demonstrate (Hayman, 1986). However, significant benefits have been noted on the farm unit basis. The UTRCA, (1989) noted in the Kintore Creek demonstration project that crop fertilizers and pesticides are held on the field reducing the amount of pollutants reaching the stream when fields were ploughed to increase surface roughness and a mulch finisher was employed. The UTRCA also noted that the ultimate method of reducing pollutant migration into streams was through a non-till system. Unfortunately, no quantitative estimates were provided to support these observations. This reflects the difficulties of collecting representative field based samples of NPS contaminants and the difficulties associated with monitoring the collective stream based response in a complex biochemical/sediment regime. Additional monitoring is currently underway.

4.5.4 *Conceptual Remediation Plan*

The assimilative capacity of the stream is used up in part by non point sources. If NPS loadings are reduced, it is evident that the municipal and industrial point sources will have more assimilative capacity available. A conceptual plan for the implementation of remediation measures within the South Nation River Watershed is outlined below.

PART 1: Characterization

1) Survey of agricultural practices and rural septic treatment systems/industrial sources, and rural subdivisions; and,

2) water quality monitoring for long term demonstration projects.

PART 2: Institutional Controls

1) Implementation of Information/education programs; and,

2) Development of Township by-laws within the SNRCA jurisdiction.

PART 3: Financial Programs

1) Proactive targeting of grants; and,

2) Cost/effectiveness analyses of remediation programs.

PART 4: Long Term Monitoring

1) Monitoring of the effectiveness of remedial measures (an extension of the PART 1: Characterization investigations); and,

2) Enforcement of violations of applicable environmental regulations.

SECTION 5

EFFLUENT TOXICITY AND DILUTION REQUIREMENTS

EFFLUENT TOXICITY AND DILUTION REQUIREMENTS

5.1 *EFFLUENT TOXICITY*

MOE is currently studying the possibility of implementing effluent toxicity criteria. For effluent to be non-lethal, concentrations of un-ionized ammonia and hydrogen sulphide must not exceed 0.1 and 0.02 mg/L, respectively. Effluent is evaluated "in the pipe" and not at stream conditions. The toxicity of effluents from the various processes given in Table 2-9 are evaluated for expected field conditions. The effluent quality from lagoon systems (without any prior treatment or aeration) is of particular concern because they are the existing primary method of wastewater treatment and effluent quality from these systems is lower than effluent from the other processes. This is particularly the case for spring discharge which is when the bulk of the treated wastewater will be released because of the high streamflows that occur in the spring.

Temperature and pH are the important variables influencing the degree of dissociation of any compound. Equation 4-1 given previously, describes the dissociation of ammonia as a function of pH and temperature.

The equations relating undissociated H_2S to total H_2S (H_2S_T) are

$$[H_2S] + [HS^-] + [S^{2-}] = [H_2S_T] \qquad (5\text{-}1)$$

and

$$\frac{[H_2S]}{[H_2S_T]} = \frac{1}{1 + \frac{K_1}{[H^+]} + \frac{K_1K_2}{[H^+]^2}} \qquad (5\text{-}2)$$

where K_1 and K_2 are dissociation constants for the first and second dissociations of H_2S, respectively.

The Van't Hoff relationship and enthalpies of substances involved (Sawyer and McCarty, 1978) describe equilibrium constants as a function of temperature. The ionic strength was assumed to be 0.0069. The extended DeBye-Hückel and Güntelberg approximations (Snoeyink and Jenkins, 1980) were used to estimate activity coefficients.

$$\frac{[H_2S]}{[H_2S_T]} = \frac{1}{1 + \frac{6.348 \times 10^{-4} e^{\frac{-2616}{T}}}{[H^+]} + \frac{2.993 \times 10^{-28} e^{\frac{4565}{T}}}{[H^+]^2}} \qquad (5\text{-}3)$$

where T is in °K

A higher temperature results in a greater amount of undissociated ammonia and a lower amount of undissociated H_2S. Hydrogen sulphide is only present for conventional lagoon treatment during spring discharge. However free ammonia can potentially present a problem for all of the treatment processes discharging at any time during the year.

Using Eqs. 4-1 and 5-3, the allowable NH_{3T} and hydrogen sulphide concentrations that would satisfy the effluent non-lethal criteria given above were calculated and are shown in Table 5-1. Spring and fall months were examined. Temperatures of lagoon and other treatment processes effluents are expected to be near the stream temperatures which were reported in the previous chapter. The low temperature for a month is the lowest 25 percentile value found for the Casselman and Plantagenet water treatment plants; the high temperature for a month is the highest of the 75 percentile values found for these two water treatment plants.

The pH in lagoons is expected to be near 7 during the winter and early spring (MOE, 1988). During the summer months the pH can rise above 8 due to algal removal of CO_2 (Metcalf & Eddy, 1991). The other treatment processes are generally expected to have an effluent pH in the range of 7 to 8 (Metcalf & Eddy, 1991). If effluent from any process is stored in a pond for seasonal discharge, the pond is equivalent to a maturation pond and algal blooms can cause the pH to rise to 10 or higher (Metcalf & Eddy, 1991).

From comparison of results in Table 5-1 to predicted effluent NH_{3T} concentrations in Table 2-9, effluent from all treatment options will satisfy the non-lethality criterion for un-ionized ammonia when their pH is 7 or less.

It is likely that effluents from an extended aeration plant with nitrification and the New Hamburg process will be compliant at all times with the ammonia criterion if their effluent is directly discharged. In the South Nation watershed effluent from any process will have to be stored in ponds. There will be times when algal activity will cause the pH to rise well above 8 and cause violation of the ammonia lethality standard.

The custom treatment process at Ault reduces its effluent pH to 7.5 or lower and at their effluent NH_{3T} concentrations the non-lethality criterion will not be violated.

Lagoon effluents have the potential to violate the ammonia criterion at times. For the temperature ranges in Table 5-1, algae activity in the fall in conventional lagoons will not be high which will reduce the likelihood of pH being above 8. Furthermore fall effluent from a conventional lagoon is highly nitrified; therefore fall lagoon discharges are highly unlikely to result in a lethal effluent with respect to ammonia. However spring discharges from a lagoon could result in some violations depending on the dynamics of algae growth and nitrification. The typical May effluent NH_{3T} concentration of 6 mg/L in Table 2-9 would result in a violation if the pH is greater than 7.68 at a temperature of 19°C.

Values for total hydrogen sulphide in lagoon effluents during the early spring discharge range from 0 to 6 mg/L. The allowable undissociated H_2S concentrations at

spring conditions are less than 0.036 and 0.18 mg/L at a pH of 7 and 8, respectively. It is necessary to provide a post aeration cell to insure the non-lethality of conventional lagoon effluent during the spring discharge period. Post aeration will strip and oxidize H_2S. Although the same phenomena will occur for un-ionized ammonia, it is assumed that the effects on ammonia will be negligible because of the slower kinetics that apply.

Table 5-1 Allowable Total Ammonia and Hydrogen Sulphide Concentrations for Non-lethal Criteria[1]

Month	Temperature[2] °C	pH	Total NH_3, mg/L		Total H_2S, mg/L	
			7	8	7	8
March	1		109	11.0	0.029	0.111
	3		92.3	9.3	0.030	0.117
April	6		73.0	7.4	0.031	0.128
	10		54.1	5.5	0.032	0.143
May	13		45.3	4.4	0.034	0.155
	17		32.8	3.4	0.035	0.173
October	10		54.1	5.5		
	13		43.5	4.4		
November	4		85.2	8.6		
	7		62.6	6.9		
December	1		109.2	11.0		
	3		92.3	9.3		

1. Un-ionized ammonia $\leq$ 0.1 mg/L; undissociated $H_2S \leq$ 0.02 mg/L.
2. The minimum monthly temperature is the minimum 25 percentile value recorded at the Casselman and Plantagenet water treatment plants. The maximum monthly temperature is the maximum 75 percentile value recorded at the Casselman and Plantagenet WTPs. (See Appendix H).

5.2 *STREAMFLOW AND DISCHARGE CONSTRAINTS*

5.2.1 *Real Time Control*

Real time control of lagoon discharges is possible but practical implementation of this strategy is difficult. The lagoon operator must be informed of daily flow variation in the receiving body of water and make adjustments in the lagoon discharge rate as necessary. Failure to make necessary flow adjustments may lead to disastrous consequences for the stream. Furthermore, the lagoon operator may be required to

know the discharge status of other nearby lagoons. Coordinating discharges on a real time basis not only requires close communication among lagoon operators but also requires system analysis to determine stream capacities at each location. It was therefore assumed that lagoon discharge rates would be constant. The river flowrate during a discharge window was assumed to be the minimum flowrate that occurs during the discharge window, i.e., river flows were equal to or greater than this value at all times during the discharge window.

The constraints mentioned above are not insurmountable. Although streamflow monitoring at critical locations should be performed on a daily basis, the decision time and reaction times in the system will be longer than one day because streamflow variation will not exhibit dramatic fluctuations in this short of a time span. Streamflow variation exhibits gradual changes and the recession curve for the spring freshet or any rise in flow is more gradual than the rising stage of the flood. This gives more planning time during the more critical periods when flows are diminishing. Furthermore there are predictive tools available for flows which provides more guidance information for optimizing discharges.

The discharge philosophy presented in the initial paragraph of this section was adopted for this study because it is the simplest and least costly approach if proven to be feasible. A more in-depth study would be required to examine the feasibility of the latter philosophy along with examination of a more sophisticated and costly system operation.

5.2.2 7Q20 and 30Q20 Flow Constraints

As shown in Chapter 3, spring 7Q20 flows are about one-half of spring 30Q20 flows; and fall 7Q20 flows are one-third to one-fourth of fall 30Q20 flows. Thus the 7Q20 flow criterion results in the most severe wastewater treatment requirements. This report will result in a prescribed discharge interval for each communities' treated waste. The 7Q20 criterion could be argued to be the most applicable criterion for the following reasons:

- Waste effluent will be discharged at an approximately constant rate.
- The longer time span used to compute 30Q20 flows allows for considerable periods of time where the streamflow drops below the average 30 day value and poses significant risks for stream fauna in particular.

The latter point is somewhat demonstrated by the lower 7 day average flows. It must also be considered that the lower 7 day flows reflect the fact that the spring flood occurrence varies from year-to-year. As a hypothetical example, assume that the spring flood curve at a particular location is always a normal curve that has a peak flow of 9 m^3/s. This spring flood curve is impressed on a constant baseline flow of 1 m^3/s. In year one the flood peak occurs in the first week of April, in year two, it occurs in the second week, in year three, the third week and in year four, the fourth week. In this scenario, the 30 day low average flow will be about the same for all

years and it will include the spring flood but 7 day low flows for each of the four weeks of April will be near the baseline flow.

Basing results on the 30 day low flow involves a measure of real time control. A fixed starting date for discharge can not be specified. The streamflow must be carefully monitored. The issue is not simply resolved by replacing the fixed date, fixed discharge interval with only specifying a fixed discharge interval for each location with the caveat that the streamflow has risen to a specified value.

Two factors militate against this rule. The stream may experience a temporary rise in flow that passes the flow criterion and then drop back to low flows until the true flood arrives. In this case without real time control, the stream deteriorates below standards if the discharge rate is not adjusted. To guard against this happening, the streamflow criterion for release of effluent can be raised to provide a higher "guarantee" that the flood is occurring. But this will cause a large amount of flow to pass which is a lost opportunity.

It is not possible to predict peculiarities of the flood progression during any given year. The operators must continually be monitoring and responding to the stream situation (including interactions among the sources) with only specifying a discharge time span with a variable starting date. Specifying a fixed interval with a fixed starting date based on an analysis of historical flow data provides not only the simplest operating rule but also the safest operating rule. With the pace of global climate change apparently accelerating, re-evaluation of flow trends on a regular basis (perhaps 5 to 10 year intervals) will show if there is a trend to changing streamflows. When a change in streamflow quantities or time of occurrence is found to be significant, discharge starting dates and intervals should be re-evaluated.

Again real time control poses a solution in the event that the conservative approach adopted in this study does not prove to be feasible.

The 30Q20 streamflow has been the historical basis for design of lagoons and their discharges in the South Nation watershed. Other constraints used in this study are conservative. Although the 30Q20 streamflows are not the most conservative measure for this parameter, they are chosen as the primary measure considering the conservative estimates of all other parameters. A stochastic analysis of the interaction of streamflow and other parameters is another useful tool that may be considered for a more complete analysis of the system.

5.3 *AVAILABLE DILUTION*

For a preliminary scan of the suitability of lagoons and other processes for treatment at existing locations of point sources it is useful to examine the dilution available and the dilution required considering the river and wastewater flowrates at each location. This preliminary assessment will define constraints and eliminate some options from consideration in the final optimization modelling exercise. It will also provide some guidance on the optimization work.

The available dilution in each discharge window was calculated according to the following equation:

$$D = \frac{Q_{r,d}}{\frac{V_w}{T_d}} \qquad (5\text{-}4)$$

where D is the dilution factor
$Q_{r,d}$ is the flow in the river during the window
V_w is the annual volume of waste
T_d is the time duration of the window

The data in Tables 3-8 and 3-9 were used to calculate dilutions available at each point source location using the 7Q20 and 30Q20 flows and durations for each of the minimum, average and maximum discharge windows during the spring and fall seasons. The flows for each point source were obtained from Table 2-8. The dilution factor for a given discharge window was calculated by discharging the total annual volume of wastewater over the duration of the window. The available dilution factors are given in Tables 5-2 and 5-3 for present conditions for the spring and fall seasons, respectively and similar results for future conditions are given in Tables 5-4 and 5-6.

Table 5-2 Spring Dilution Factors for Present Conditions

	7Q20			30Q20		
	Windows			Windows		
Point Source	Min	Ave	Max	Min	Ave	Max
Casselman	66.5	61.5	48.6	146.1	132.1	102.7
Chesterville	36.2	33.4	26.4	79.4	71.8	55.8
Embrun	14.9	13.8	10.9	32.7	29.6	23.0
Maxville	0.3	0.3	0.2	0.7	0.7	0.5
Plantagenet	171.2	158.1	125.1	375.9	340.0	264.4
Russell	24.0	22.2	17.5	52.7	47.7	37.1
Spencerville	48.0	44.3	35.1	105.4	95.3	74.1
St. Isidore	15.4	14.2	11.3	33.8	30.6	23.8
Williamsburg	2.5	2.3	1.8	5.5	5.0	3.9
Winchester	0.1	0.1	0.1	0.3	0.3	0.2
Ault	0.4	0.3	0.3	0.8	0.7	0.6
Nestle	47.4	43.8	34.6	104	94.1	73.2

The dilution factor is directly proportional to the quantity of effluent that is discharged and the length of time for which the discharge occurs. Dilution factors vary irregularly with the time period over which the discharge occurs because of the flow variation in the stream.

It is observed that the minimum window provides the highest dilution factors at all locations for both the 7Q20 and 30Q20 flows during the spring (Tables 5-2 and 5-4). The relative decrease in 7Q20 and 30Q20 flows as the window or discharge period is increased is greater than the increase in duration of the windows which leads to the overall decrease in the dilution factor. Advantage of the increasing dilution with decreasing period of discharge can be taken by staggering discharges.

Table 5-3 Fall Dilution Factors for Present Conditions

	7Q20			30Q20		
	Windows			Windows		
Point Source	Min	Ave	Max	Min	Ave	Max
Casselman	1.6	2.0	1.2	6.4	8.3	6.0
Chesterville	0.8	1.1	0.7	3.5	4.5	3.2
Embrun	0.3	0.5	0.3	1.4	1.9	1.3
Maxville	0.0	0.0	0.0	0.0	0.0	0.0
Plantagenet	4.0	5.2	3.1	16.3	21.5	15.3
Russell	0.6	0.7	0.4	2.3	3.0	2.1
Spencerville	1.1	1.5	0.9	4.6	6.0	4.3
St. Isidore	0.4	0.5	0.3	1.5	1.9	1.4
Williamsburg	0.1	0.1	0.0	0.2	0.3	0.2
Winchester	0.0	0.0	0.0	0.0	0.0	0.0
Ault	0.0	0.0	0.0	0.0	0.1	0.0
Nestle	1.1	1.4	0.9	4.5	5.9	4.2

For the fall (Tables 5-3 and 5-5), the average window provides the largest dilution factor for both 7Q20 and 30Q20 flows because the relative flow decrease in moving from the minimum to the average window is exceeded by the relative increase in duration of the average window compared to the minimum window. Because the minimum window is contained within the average and maximum windows and dilutions decrease from the average to the minimum window, staging discharges will be most beneficial through the maximum and average windows.

The other significant observation to be made from Tables 5-2 - 5-5 is the very large decrease in available dilution in the fall compared to the spring. Available dilution

factors decrease by factors near 40 and 20 for 7Q20 and 30Q20 flows, respectively. The spring freshet is the primary period for discharge of effluent.

The available dilutions complete the first step toward describing the system constraints. The available dilutions consider that the total volume of waste is discharged during the window under consideration. There is opportunity to augment the available dilution factors by discharging a portion of the effluent during both spring and fall which will be addressed below.

There is, in general, not a large difference between the available dilution factors for present and future conditions except for the communities of Russell and Embrun which are planned to experience large population growth over the planning period of time.

Table 5-4 Spring Dilution Factors for Future Conditions

	7Q20 Windows			30Q20 Windows		
	Min	Ave	Max	Min	Ave	Max
Bourget	19.1	17.7	14.0	42.0	38.0	29.5
Casselman	25.7	23.7	18.8	56.5	51.1	39.7
Chesterville	33.6	31.0	24.5	73.7	66.7	51.8
Crysler	57.3	52.9	41.8	125.7	113.7	88.4
Embrun	4.6	4.2	3.3	10.0	9.0	7.0
Fournier	11.9	11.0	8.7	26.1	23.6	18.4
Maxville	0.3	0.3	0.2	0.7	0.7	0.5
Moose Creek	2.8	2.6	2.0	6.1	5.5	4.3
Plantagenet	151.8	140.1	110.8	333.1	301.3	234.3
Russell	5.4	4.9	3.9	11.8	10.6	8.3
Spencerville	35.3	32.5	25.8	77.5	70.1	54.5
St. Albert[1]	105.0	96.9	76.7	230.4	208.4	162.1
St. Isidore	11.1	10.2	8.1	24.3	22.0	17.1
Williamsburg	2.3	2.1	1.7	5.0	4.5	3.5
Winchester	0.1	0.1	0.1	0.2	0.2	0.2
Ault	0.4	0.4	0.3	0.9	0.9	0.7
Nestle	14.3	13.2	10.5	31.5	28.5	22.1

1. The dilution available at St. Albert is the same for both the community of St. Albert and the industry of St. Albert Cheese.

Table 5-5 Fall Dilution Factors for Future Conditions

	7Q20 Windows			30Q20 Windows		
	Min	Ave	Max	Min	Ave	Max
Bourget	0.4	0.6	0.3	1.8	2.4	1.7
Casselman	0.6	0.8	0.5	2.4	3.2	2.3
Chesterville	0.8	1.0	0.6	3.2	4.2	3.0
Crysler	1.3	1.7	1.0	5.5	7.2	5.1
Embrun	0.1	0.1	0.1	0.4	0.6	0.4
Fournier	0.3	0.4	0.2	1.1	1.5	1.1
Maxville	0.0	0.0	0.0	0.0	0.0	0.0
Moose Creek	0.1	0.1	0.1	0.3	0.3	0.2
Plantagenet	3.5	4.6	2.8	14.5	19.0	13.6
Russell	0.1	0.2	0.1	0.5	0.7	0.5
Spencerville	0.8	1.1	0.7	3.3	4.4	3.2
St. Albert[1]	2.5	3.2	1.9	10.0	13.2	9.4
St. Isidore	0.3	0.3	0.2	1.1	1.4	1.0
Williamsburg	0.1	0.1	0.0	0.2	0.3	0.2
Winchester	0.0	0.0	0.0	0.0	0.0	0.0
Ault	0.0	0.0	0.0	0.0	0.1	0.0
Nestle	0.3	0.4	0.3	1.4	1.8	1.3

1. The dilution available at St. Albert is the same for both the community of St. Albert and the industry of St. Albert Cheese.

5.4 *FEASIBILITY OF MEETING PWQO STREAM STANDARDS AT 7Q20 AND 30Q20 STREAMFLOWS*

The instream PWQO for un-ionized ammonia and hydrogen sulphide are 0.02 and 0.002 mg/L, respectively. These limits will impose a dilution requirement on effluents from the processes listed in Table 2-9.

To determine dilution requirements for these substances it is necessary that stream conditions be considered. The 25 and 75 percentile stream temperatures and pHs from the Casselman and Plantagenet water treatment plant data were agreed to be reasonable extremes at which to conduct the analysis. Complete mixing between the effluent and the receiving water were specified in the TOR as being adequate to conduct this phase of the analysis. The mass balance relation that applies to

determine the required dilution considering waste and streamflow and concentrations is

$$\frac{Q_r}{Q_w} = \frac{C_w - C_m}{C_m - C_r} \qquad (5\text{-}5)$$

where Q_r is the river flow immediately upstream of the mixing point
Q_w is the wastewater flow from the point source
C_w is the concentration in the wastewater
C_r is the concentration in the river
C_m is the concentration in the river after mixing with the point source wastewater

This equation was used in conjunction with other equations noted in Sections 4.2.1 and 5.1 for hydrogen sulphide and un-ionized ammonia to determine the dilution requirements for these compounds in Table 5-6. Stream temperature and pH values are also required to determine the dilution requirements for these two compounds. Temperatures were obtained from Table 5-1 and are reproduced in Table 5-6. The lower and upper pH values for each month were determined in the same manner as the lower and upper temperatures.

It must be emphasized that dilution factors in Table 5-6 are the **minimum** required dilutions because they were derived assuming that background river concentrations were not being elevated by the simultaneous discharge of more than one point source.

5.4.1 Hydrogen Sulphide

The concentration of un-ionized H_2S at any pH and temperature is calculated with Eq. 5-3 for the specified effluent H_2S_T concentrations given in Table 2-9 which determines C_w in Eq. 5-5. Then Eq. 5-5 is used to calculate the required dilution with C_m at 0.002 mg/L and $C_r = 0$ (which assumes that there is no background concentration of H_2S in the river).

The dilution requirements in Table 5-6 must be related to the dilution capacities available at each point source in Tables 5-2 - 5-5 The discharge windows for the spring begin in March and the dilution capacity required for H_2S at 3 mg/L ranges from 591 to 1104. Even at 1 mg/L H_2S_T, the dilution requirement ranges from 192 to 367. No municipal point source has a dilution capacity available considering either 7Q20 or 30Q20 flows to meet the March dilution requirement even if the dilution capacity is doubled due to a seasonal discharge which would double the spring dilution capacity. This conclusion applies to both present and future conditions. April dilution requirements for H_2S are also severe, ranging up to 894 under the worst case conditions. All lagoons that will discharge during the spring must have a post aeration cell to insure that the H_2S stream toxicity standard is not violated. The other alternative is to install a more advanced treatment process that removes H_2S.

Table 5-6 Dilution Required[1] for Effluent Ammonia and Hydrogen Sulphide Concentrations to meet PWQO

Month	pH[2]	temp[2] °C	DILUTION						
			NH_3 Concentration, mg/L					H_2S_T, mg/L	
			14	10	6	5	4	3	1
Mar	6.9	1	0.0	0.0	0.0	0.0	0.0	1104.0	367.3
	7.5	1	1.0	0.5	0.0	0.0	0.0	615.5	204.5
	6.9	3	0.0	0.0	0.0	0.0	0.0	1081.5	359.8
	7.5	3	1.4	0.7	0.0	0.0	0.0	590.6	196.2
Apr	7.1	6	0.2	0.0	0.0	0.0	0.0	893.5	297.2
	7.9	6	7.3	4.9	2.5	1.9	1.3	283.6	93.9
	7.1	10	0.6	0.2	0.0	0.0	0.0	845.1	281.0
	7.9	10	10.5	7.2	3.9	3.0	2.2	254.3	84.1
May	7.6	13	5.8	3.9	1.9	1.4	0.9	405.2	134.4
	8.1	13	25.2	17.6	10.0	8.1	6.2	156.6	51.5
	7.6	17	8.3	5.6	2.9	2.3	1.6	368.9	122.3
	8.1	17	37.7	26.5	15.3	12.5	9.7	139.7	45.9
Oct	7.6	10	4.4	2.9	1.3	0.9	0.5		
	7.9	10	10.5	7.2	3.9	3.0	2.2		
	7.6	13	5.8	3.9	1.9	1.4	0.9		
	7.9	13	13.8	9.5	5.3	4.2	3.1		
Nov	7.6	4	2.4	1.4	0.4	0.2	0.0		
	8	4	8.0	5.4	2.8	2.1	1.5		
	7.6	7	3.3	2.0	0.8	0.5	0.2		
	8	7	10.6	7.3	3.9	3.1	2.2		
Dec	7.4	1	0.6	0.2	0.0	0.0	0.0		
	7.9	1	4.3	2.8	1.2	0.9	0.5		
	7.4	3	0.9	0.4	0.0	0.0	0.0		
	7.9	3	5.4	3.5	1.7	1.2	0.8		

1. Assuming a background concentration for NH_{3T} of 0.18 mg/L and a background concentration for H_2S_T of 0 mg/L.
2. The minimum monthly temperature or pH is the minimum 25 percentile value recorded at Casselman or Plantagenet water treatment plants. The maximum monthly temperature or pH is the maximum 75 percentile value recorded at above mentioned WTPs. (See Appendix H).

Significant H_2S concentrations occur in spring because of ice cover and ensuing anaerobic conditions for a prolonged period of time during winter months. H_2S will not be problematic for lagoons during the fall.

5.4.2 *Un-ionized Ammonia*

The dilutions required to meet the PWQO standard for un-ionized ammonia for concentrations of NH_{3T} corresponding to expected effluent concentrations for the various wastewater treatment processes in Table 2-9 are also shown in Table 5-6. The equation (approved by MOE) used to determine the speciation of un-ionized and ionized ammonia is Eq. 4-1.

From Eq. 5-6, the ratio of un-ionized ammonia to total ammonia can be obtained.

$$\frac{[NH_3]}{[NH_{3T}]} = \frac{X}{1 + X} \qquad (5\text{-}6)$$

where X is the ratio of $[NH_3]/[NH_4^+]$ obtained from Eq. 4-1

Note that a background concentration of 0.18 mg/L NH_{3T} was assumed to be present in the receiving waters. All NH_{3T} concentrations were adjusted to un-ionized ammonia concentrations using Eqs. 4-1 and 5-6 which were solved with extreme values of temperature and pH (25 and 75 percentile values from Casselman and Plantagenet WTP data for both parameters) for each month of interest.

The expected effluent NH_{3T} concentrations for lagoons are specified in Table 2-9. Effluent specifications to be used for the industries have been discussed in Sections 2.3 and 2.7.

The ability of lagoons and industries to meet the PWQO for un-ionized ammonia is evaluated in Tables 5-7 and 5-8 for complete mixed conditions at present and future conditions, respectively. These tables were constructed by taking the maximum available dilutions from Tables 5-2 and 5-4 (spring conditions) and comparing these results with the maximum required dilutions during the months of March and April. April is the month which comprises most of the discharge windows. A few windows occur during March and only one window (the spring 7Q20 maximum window) extends into the month of May. The April dilution requirement is much larger than the March dilution requirement primarily because of the higher temperatures that occur during April.

Russell and Embrun were evaluated separately and using a combined treatment facility which decreases the overall available dilution capacity. It is probable that there will be need for considerable overlap between the discharge periods for these communities to take advantage of the highest flows in the stream.

Table 5-7 Ability of Lagoon Processes at Point Source Locations to Meet the PWQO for Ammonia for Current Conditions

	Available Maximum Dilution		Maximum Required Dilution	
Point Source	7Q20	30Q20	Mar. 1.4	Apr. 10.5
Casselman	66.5	146.1	Y[2]	Y
Chesterville	36.2	79.4	Y	Y
Embrun	14.9	32.7	Y	Y
Maxville	0.3	0.7	N[2]	N
Plantagenet	171.2	375.9	Y	Y
Russell	24.0	52.7	Y	Y
Spencerville	48.0	105.4	Y	Y
St. Isidore	15.4	33.8	Y	Y
Williamsburg	2.5	5.5	Y	N
Winchester	0.1	0.3	N	N
Embrun and Russell[3]	9.2	20.2	Y	N7[2]
Industries			0.0[4]	3.9[4]
Ault	0.4	0.8	Y	N
Nestle	47.4	104	Y	Y

1. The lagoon effluent dilution requirement is based on 14 mg/L NH_{3T} for March and April.
2. Y- Both 7Q20 and 30Q20 dilution requirements are equalled or exceeded; N- neither the 7Q20 or 30Q20 dilution requirement is available; N7- the 7Q20 dilution requirement is not met.
3. The available dilution is for both communities discharging at Embrun.
4. The maximum required dilution for Ault is based on an effluent NH_{3T} concentration of 6.1 mg/L as specified in the C of A. The maximum required dilution for Nestle is based on an effluent NH_{3T} concentration of 6.1 mg/L as discussed in the text.

Table 5-8 Feasibility of Lagoon Processes[1] at Point Source Locations to Meet the PWQO for Ammonia for Future Conditions

	Available Maximum Dilution		Maximum Required Dilution	
Point Source	7Q20	30Q20	Mar. 1.4	Apr. 10.5
Bourget	19.1	42.0	Y[2]	Y
Casselman	58.1	127.5	Y	Y
Chesterville	33.6	73.7	Y	Y
Crysler	57.3	125.7	Y	Y
Embrun	4.6	10.0	Y	N
Fournier	11.9	26.1	Y	Y
Maxville	0.3	0.7	N[2]	N
Moose Creek	2.8	6.1	Y	N
Plantagenet	151.8	333.1	Y	Y
Russell	5.4	11.8	Y	N7[2]
Spencerville	35.3	77.5	Y	7
St. Albert	105.0	230.4	Y	Y
St. Isidore	11.1	24.3	Y	Y
Williamsburg	2.3	5.0	Y	N
Winchester	0.1	0.2	N	N
Embrun and Russell[3]	3.0	6.4	Y	N
Industries			0.0[4]	3.9[4]
Ault	0.4	0.9	Y	N
Nestle	47.4	104	Y	Y
St. Albert Cheese	105.0	230.4	Y	Y

1. Lagoon effluent dilution requirement in Mar. is based on 14 mg/L NH_{3T}.
2. Y- Both 7Q20 and 30Q20 dilution requirements are equalled or exceeded; N- neither the 7Q20 or 30Q20 dilution requirement is available; N7- the 7Q20 dilution requirement is not available.
3. The available dilution is for both communities discharging at Embrun.
4. The maximum required dilution for all industries is based on an effluent NH_{3T} concentration of 6.1 mg/L.

Results in Tables 5-7 and 5-8 are derived assuming that each point source was discharging individually without other sources elevating the background concentration as explained in Section 5.4. Sources with an "N" or "N7" designation must definitely apply remedial measures of effluent treatment upgrade, relocation of discharge point or a combination of these. These results were used as the starting point for the final analysis presented in Chap. 6. Further analysis in Chap. 6 with all sources discharging will also result in the need for remedial measures at some locations which are indicated in Tables 5-7 and 5-8 as satisfactory when the individual point source is the only effluent source.

It is not necessary to conduct a mixing zone analysis when complete mixed conditions will not provide the desired concentrations. However, when complete mixed conditions show that a lagoon or any other process is feasible, a mixing zone analysis may show that the process is not feasible. Mixing zone analyses are discussed in Chapter 7.

In the cases of Maxville and Winchester, the dilution available at either 7Q20 or 30Q20 flows in their receiving streams is insufficient to meet the PWQO for un-ionized ammonia at present or future conditions in either month.

A number of communities and one industry (Ault) can not meet the April dilution requirement at either 7Q20 or 30Q20 flows.

It should be noted at this point that even if a discharger can release its waste in compliance with the PWQO for ammonia, it is possible that the dissolved oxygen constraint will not be met. This will be discussed further in this report.

5.4.3 Maximum Potential for Fall Lagoon Discharges

There exists an opportunity for some of the communities to discharge a portion of the annual waste volume during the fall. This will improve the spring situation for some of the point sources. For preliminary analysis, the **maximum** waste volumes that can be discharged during the fall in each window for lagoon treatment for all communities were calculated, i.e., it was assumed that no other communities were simultaneously discharging their effluents. For lagoons discharging during the fall effluent NH_{3T} concentrations are 4 mg/L. The fall discharge windows are confined to the months of November and December. The constraining month is November with a pH of 8.0 and a temperature of 7°C, which substituted into Eq. 4-1 results in an un-ionized to ionized ammonia ratio of 0.0146.

The industries were assumed to be discharging 6.1 mg/L NH_{3T} in accordance with effluent specifications described earlier.

The analysis was performed for communities and industries that are currently discharging as point sources. The future scenario will be addressed in the modelling exercise to follow. Tables 5-9 and 5-10 present fall discharge volumes for 7Q20 and 30Q20 flows, respectively. The volumes discharged are also expressed as a fraction of the total waste volume for the source. The maximum volumes are discharged in the average windows for both 7Q20 and 30Q20 flows.

From Table 5-9, on a 7Q20 basis Russell and Embrun can discharge a maximum of 15% (using the average window) of their annual waste volume during the fall period. However on a 30Q20 basis, these communities could discharge 60% of their annual waste volume during the fall period.

For Maxville and Winchester, the fraction of their waste volumes that could be discharged during the average window in the fall which provides the maximum allowable waste volume discharge, will not reduce the volumes to be discharged during the spring to levels that will allow them to meet the ammonia standard. It is definitely concluded that these communities must choose another alternative. The other communities will require further modelling analysis to arrive at definite conclusions.

Table 5-9 Maximum Potential Volumes for 7Q20 Fall Discharge[1]

MUNICIPALITY	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	Annual Volume m^3	Minimum Window (18 d)			Average Window (34 d)			Maximum Window (51 d)		
		Stream Flow m^3/s	Volume Discharged m^3	Ratio (3)/(1)	Stream Flow m^3/s	Volume Discharged m^3	Ratio (6)/(1)	Stream Flow m^3/s	Volume Discharged m^3	Ratio (9)/(1)
Bourget	230200	0.066	46575	0.20	0.046	60673	0.26	0.018	36404	0.16
Casselman	365000	0.365	257019	0.70	0.252	334814	0.92	0.101	200888	0.55
Chesterville	296015	0.161	113234	0.38	0.111	147507	0.50	0.044	88504	0.30
Crysler	219000	0.189	132676	0.61	0.130	172834	0.79	0.052	103700	0.47
Embrun	474500	0.106	74777	0.16	0.073	97411	0.21	0.029	58446	0.12
Fournier	50900	0.009	6409	0.13	0.006	8349	0.16	0.003	5010	0.10
Maxville	59860	0.000	214	0.00	0.000	278	0.00	0.000	167	0.00
Moose Creek	101600	0.004	2991	0.03	0.003	3896	0.04	0.001	2338	0.02
Plantagenet	225205	0.580	408068	1.81	0.400	531583	2.36	0.160	318950	1.42
Russell	193085	0.070	49032	0.25	0.048	63873	0.33	0.019	38324	0.20
Spencerville	51100	0.037	25958	0.51	0.025	33815	0.66	0.010	20289	0.40
St. Albert	138400	0.218	153720	1.11	0.151	200248	1.45	0.060	120149	0.87
St. Isidore	127020	0.029	20724	0.16	0.020	26997	0.21	0.008	16198	0.13
Williamsburg	64094	0.002	1709	0.03	0.002	2227	0.03	0.001	1336	0.02
Winchester	629625	0.001	855	0.00	0.001	1113	0.00	0.000	668	0.00
Emb + Russ	667585	0.106	74777	0.11	0.073	97411	0.15	0.029	58446	0.09
Ault[2]	193800	0.001	416	0.00	0.001	542	0.00	0.000	325	0.00
Nestles[2]	225900	0.161	62962	0.28	0.111	82020	0.36	0.044	49212	0.22
St.Albert Chs	22600	0.218	85474	3.78	0.151	111346	4.93	0.060	66807	2.96

1. Based on the maximum un-ionized to ionized ammonia ratio of 0.0146 (which occurs at extreme conditions of pH = 8.0 and temperature of 7°C in November). The background concentration of NH_{3T} was assumed to be 0.18 mg/L.
2. All industries were assumed to have 6.1 mg/L NH_{3T} in their effluents.
3. The annual volume is the annual volume for present conditions except for the communities of Bourget, Crysler, Moose Creek and St. Albert and the industry of St. Albert Cheese.
4. A streamflow of 0.000 indicates that the streamflow was < 0.0005.

Table 5-10 Maximum Potential Volumes for 30Q20 Fall Discharge[1]

MUNICIPALITY	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	Annual Volume m^3	Minimum Window (22 d)			Average Window (32 d)			Maximum Window (43 d)		
		Stream Flow m^3/s	Volume Discharged m^3	Ratio (3)/(1)	Stream Flow m^3/s	Volume Discharged m^3	Ratio (6)/(1)	Stream Flow m^3/s	Volume Discharged m^3	Ratio (9)/(1)
Bourget	230200	0.221	190086	0.83	0.200	249767	1.08	0.106	178071	0.77
Casselman	365000	1.220	1048959	2.87	1.102	1378299	3.78	0.585	982659	2.69
Chesterville	296015	0.537	462135	1.56	0.485	607231	2.05	0.258	432925	1.46
Crysler	219000	0.630	541483	2.47	0.569	711491	3.25	0.302	507258	2.32
Embrun	474500	0.355	305183	0.64	0.321	401001	0.85	0.170	285894	0.60
Fournier	50900	0.030	26159	0.51	0.027	34372	0.68	0.015	24505	0.48
Maxville	59860	0.001	872	0.01	0.001	1146	0.02	0.000	817	0.01
Moose Creek	101600	0.014	12207	0.12	0.013	16040	0.16	0.007	11436	0.11
Plantagenet	225205	1.937	1665430	7.40	1.750	2188322	9.72	0.928	1560165	6.93
Russell	193085	0.233	200113	1.04	0.210	262942	1.36	0.112	187465	0.97
Spencerville	51100	0.123	105942	2.07	0.111	139205	2.72	0.059	99246	1.94
St. Albert	138400	0.730	627370	4.53	0.659	824344	5.96	0.350	587716	4.25
St. Isidore	127020	0.098	84579	0.67	0.089	111135	0.87	0.047	79233	0.62
Williamsburg	64094	0.008	6976	0.11	0.007	9166	0.14	0.004	6535	0.10
Winchester	629625	0.004	3488	0.01	0.004	4583	0.01	0.002	3267	0.01
Emb + Russ	667585	0.355	305183	0.46	0.321	401001	0.60	0.170	285894	0.43
Ault	193800	0.004	1697	0.01	0.003	2230	0.01	0.002	1590	0.01
Nestles	225900	0.537	256965	1.14	0.485	337643	1.49	0.258	240723	1.07
St. Albert Chs	22600	0.730	348842	15.44	0.659	458367	20.28	0.350	326793	14.46

1. Based on the maximum un-ionized to ionized ammonia ratio of 0.0146 (which occurs at extreme conditions of pH = 8.0 and temperature of 7°C in November). The background concentration of NH_{3T} was assumed to be 0.18 mg/L.
2. All industries were assumed to have 6.1 mg/L NH_{3T} in their effluents.
3. The annual volume is the annual volume for present conditions except for the communities of Bourget, Crysler, Moose Creek and St. Albert and the industry of St. Albert Cheese.
4. A streamflow of 0.000 indicates that the streamflow was < 0.0005.

SECTION 6

WASTEWATER ALLOCATION OPTIMIZATION

WASTEWATER ALLOCATION OPTIMIZATION

6.1 *ALTERNATIVES FOR LAGOONS THAT ARE UNABLE TO MEET PWQO*

It has been demonstrated in the previous chapter that some communities will be unable to satisfy receiving water quality criteria at least for ammonia. The alternatives for a community that can not meet the PWQO at current population levels for one or more of the parameters are as follows:

- Stage the discharge
- Discharge some of the waste during the fall
- Install a more advanced wastewater treatment facility
- Pipe the effluent from the wastewater facility to a location where suitable dilution capacity exists
- Water conservation
- A combination of the above

In addition to the above an alternative for communities at future populations is to limit their population growth.

DO concentration fluctuations (discussed below) in general, do not violate the PWQO criterion. Ammonia is the most problematic substance to be controlled. For the spring months of March, April and May the maximum allowable NH_{3T} concentrations in the receiving waters are 5.85, 1.38 and 0.54 mg/L, respectively using the extreme values of temperature and pH (from the Casselman and Plantagenet water treatment plants data) which produce the maximum amount of un-ionized ammonia in the streams during each month (see Table 5-1 for monthly extreme temperatures and pH values). For November and December, the corresponding concentrations for NH_{3T} are 1.37 and 2.34 mg/L, respectively.

On a window basis, the maximum allowable NH_{3T} concentrations in the streams are, on a 30Q20 basis, 2.52, 2.52 and 2.25 mg/L for the spring maximum, average and minimum windows, respectively using the extreme values of temperature and pH from the Casselman and Plantagenet water treatment plants data (see Appendix H). On a 7Q20 basis, the corresponding concentrations are 1.11, 1.48 and 2.52 mg/L for the spring maximum, average and minimum windows.

6.1.1 *Routing the Lagoon Discharges to Other Locations*

Some upper reaches of tributary streams (e.g., McMartin Drain, Moose Creek and Henderson Drain) may not have sufficient flow to dilute effluent concentrations to the PWQO. The communities which discharge to these streams are usually the most upstream communities discharging to the stream and are therefore not influenced by

wastewaters from other communities. Some of these discharge points must be moved downstream to a point where streamflow is sufficient to provide required dilution.

This principle must be adhered to regardless of the type of treatment in place. The more advanced wastewater treatment processes which produce effluents with lower concentrations of pollutants would not have as great a dilution requirement. This practically means that the discharge point would not have to be moved as great a distance from the community or industry, if it would have to be moved at all.

6.1.2 Staging Effluent Discharges

In the fall, most communities cannot discharge any waste without violating the in-stream PWQO for DO. If fact, even with no one discharging, oxygen levels could still drop below PWQO levels. For this reason, it was assumed that no discharge would occur in the fall. Due to an unfavourable combination of low flows, high pH and temperatures, fall discharges are not recommended. The BOD and DO percent saturation profiles for the 30Q20 maximum fall window are shown in Fig. 6-1. It can be seen on this figure that even with zero discharge, the percent DO saturation is very close to or lower than the PWQO of 47%.

Staging the spring lagoon discharge for the communities that can not meet the ammonia PWQO on the basis described in the previous chapter can be eliminated immediately as an alternative that will by itself resolve the ammonia dilution requirement. The communities have been evaluated separately (i.e., no overlapping effects from other communities waste discharges) at the **maximum** dilution in Chapter 5. Moving the discharge to another window for a community will only result in less dilution capacity and further deteriorate the stream condition.

Staging discharges of different point sources according to the windows concept developed in Chapter 3 will lessen the need for advanced treatment or other measures such as piping the effluent to a new location. However, the constraint that a discharge rate will be constant for individual dischargers must be observed throughout this exercise. In general the principle that must be followed to use the assimilative capacity of the stream to the maximum extent is as follows: The maximum window is examined first with all streamflows set appropriately. The furthest upstream point source is examined to see if the allowable discharge rate at this location will result in a volume equal to or greater than the volume to be discharged. When the allowable discharge rate is satisfactory, the discharge rate is set to the constant rate that discharges the total volume of effluent over the duration of the window. The procedure is repeated moving downstream.

The average window is then examined with streamflows at each location adjusted to values corresponding to this window and discharge rates of those sources determined in the maximum window analysis. The procedure in the average window is exactly the same as in the maximum window.

Finally the minimum window is examined in exactly the same manner as its predecessors after setting discharge flow rates of those sources discharging in the

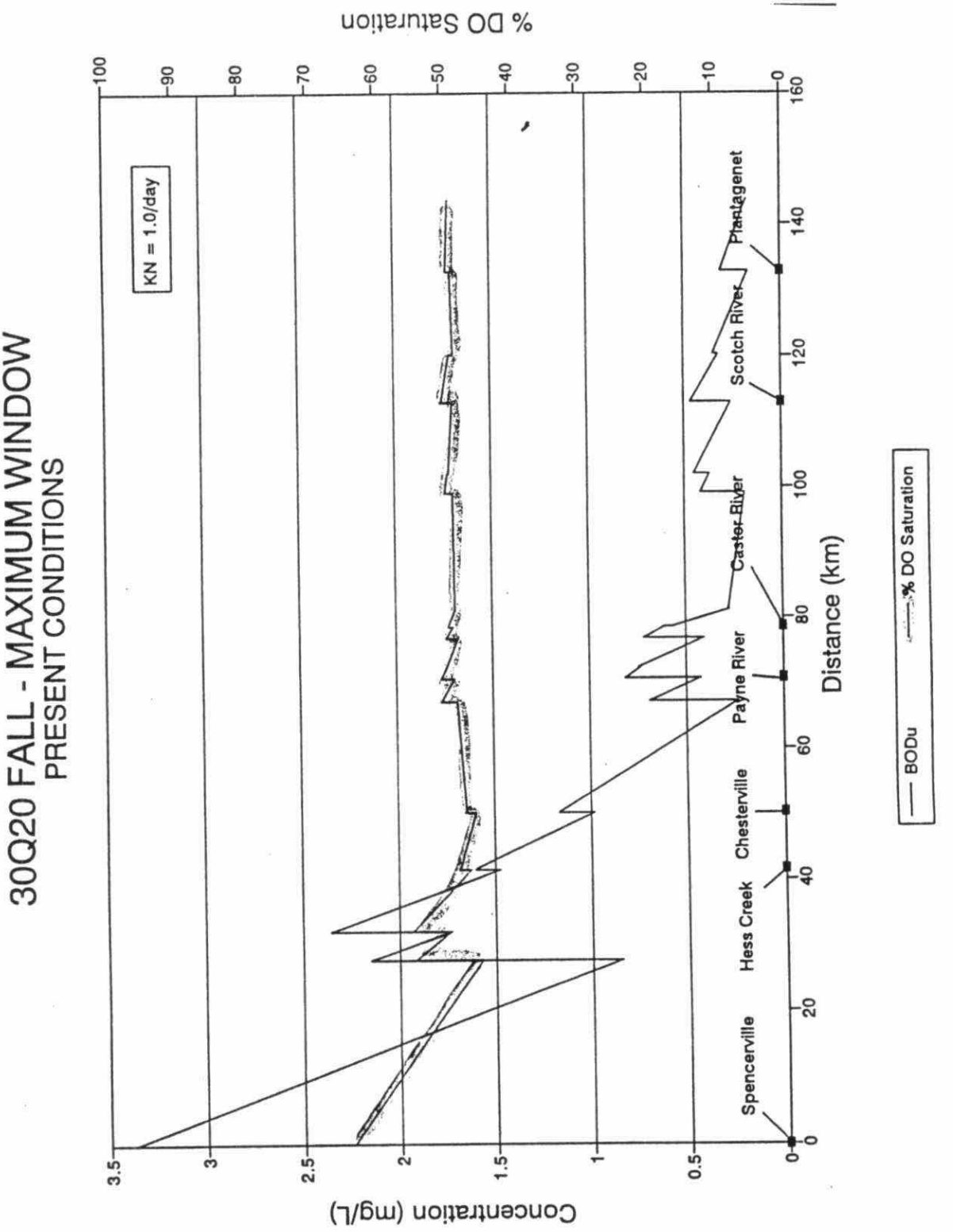

Figure 6-1 Fall 30Q20 Maximum Window DO and BOD Profile using Coefficient Set 3 (no discharges)

maximum and average windows. All three spring windows are examined because this approach makes better utilization of the flood hydrograph. At the limit, an infinite number of windows would be the equivalent of real-time control. These concepts will be expanded upon after the models are explained below.

6.2 DISSOLVED OXYGEN ANALYSIS

The South Nation River and its tributaries are judged to be warm water bodies which have a PWQO minimum DO saturation of 47% except at a temperature of 25°C where the saturation limit is 48%. The phenomena to be modelled for DO analysis are BOD exertion, nitrification of ammonia and reaeration.

The phenomena influencing BOD exertion and ammonia transformation are diverse and complex. Sediment oxygen demand (SOD), speciation of soluble and suspended BOD, volatilization and microbial synthesis are some of the phenomena influencing BOD exertion. Ammonia is involved in most synthesis and degradation reactions and it is subject to a number of chemical reactions as well as volatilization. The general approach is to lump phenomena together and choose conservative coefficients. A more sophisticated breakdown of the phenomena into their individual processes requires an extensive database which is not available. It was deemed reasonable to consider SOD in the fall because of the likelihood of deposition of organic matter throughout the summer period. Sediment decay during the winter and high streamflows in the spring were expected to remove significant quantities of sediment oxygen demanding material and render SOD negligible during the spring period.

For ammonia, the primary biological reactions involve transformation of ammonia to nitrite by *Nitrosomonas*, followed by the conversion of nitrite to nitrate by *Nitrobacter* species. The first reaction is generally limiting. Ammonia transformation will be modelled as a single step process. From a toxicity perspective, transformation of ammonia to nitrite is the most important reaction because the potential for un-ionized ammonia formation is reduced. From an oxygen exertion perspective, assuming that the overall nitrification process occurs at the rate of the first (nitrite) transformation results in a slightly higher rate of oxygen consumption, which is conservative.

The basic differential equation governing DO is then

$$\frac{dD}{dt} = k_1 L + 3.76 k_N [NH_{3T}] - k_2 D + s \qquad (6\text{-}1)$$

where D is the DO deficit (difference between saturation concentration and actual concentration)
t is time
k_1 is BOD exertion coefficient
L is concentration of BOD
k_N is rate of nitrification
k_2 is reaeration coefficient

s is rate of SOD

The integrated equation is

$$D = \frac{k_1 L_o}{k_2 - k_1}\left[e^{-k_1 t} - e^{-k_2 t}\right] + \frac{3.76 k_N N_o}{k_2 - k_N}\left[e^{-k_N t} - e^{-k_2 t}\right] + \frac{s}{k_2}\left(1 - e^{-k_2 t}\right) + D_o e^{-k_2 t} \quad (6\text{-}2)$$

where L_o is initial value of ultimate BOD
N_o is initial value of NH_{3T}
D_o is initial value of DO deficit

All kinetic coefficients are highly influenced by temperature. The standard modified Arrhenius relation is used to describe the effect of temperature on reaction coefficients:

$$k_T = k_{20}\theta^{(T-20)} \quad (6\text{-}3)$$

where T is in °C
Θ is a constant over a specified temperature interval

As noted in Chapter 4, the river data available are not suitable for calibration of a model. A literature search was initiated to obtain coefficient values for rivers similar to the South Nation. Tetra Tech (1978) has made a comprehensive review of rate coefficients for river, lake and estuary modelling.

6.2.1 *Reaeration Coefficient*

There are a number of different formulations for the reaeration coefficient. Turbulence (velocity) and depth are primary variables in most formulations. At low flows, depth in the South Nation River ranges from 0.6 - 1.3 m. For 7Q20 and 30Q20 flows the volumetric flowrate ranges between < 1 - 50 m^3/s and the velocity ranges from 0.1 - 0.4 m/s. From a survey of tables in Tetra Tech (1978) of in situ measured k_2 values for rivers with flow characteristics similar to the South Nation River, k_2 ranged from 0.07 to 5.0 d^{-1}.

The work of Covar (1976) provided a generalized unifying approach to many equations presented for the reaeration coefficient. The widely used expressions of O'Connor and Dobbins (1958) and Churchill et al. (1962) were solved for the line along which they meet. The family of curves resulting from these expressions and the line along which they give equivalent results are shown in Fig. 6-2. The field data from the work of these two groups of researchers generally fell into the zones for which their equations are graphed. It is seen that the two formulations are consistent. Covar (1976) also incorporated the work of Owens et al. (1964) which is consistent with the other two groups for very shallow streams. Tetra Tech (1978) recommended that this plot be used when stream measurements of the reaeration coefficient are unavailable.

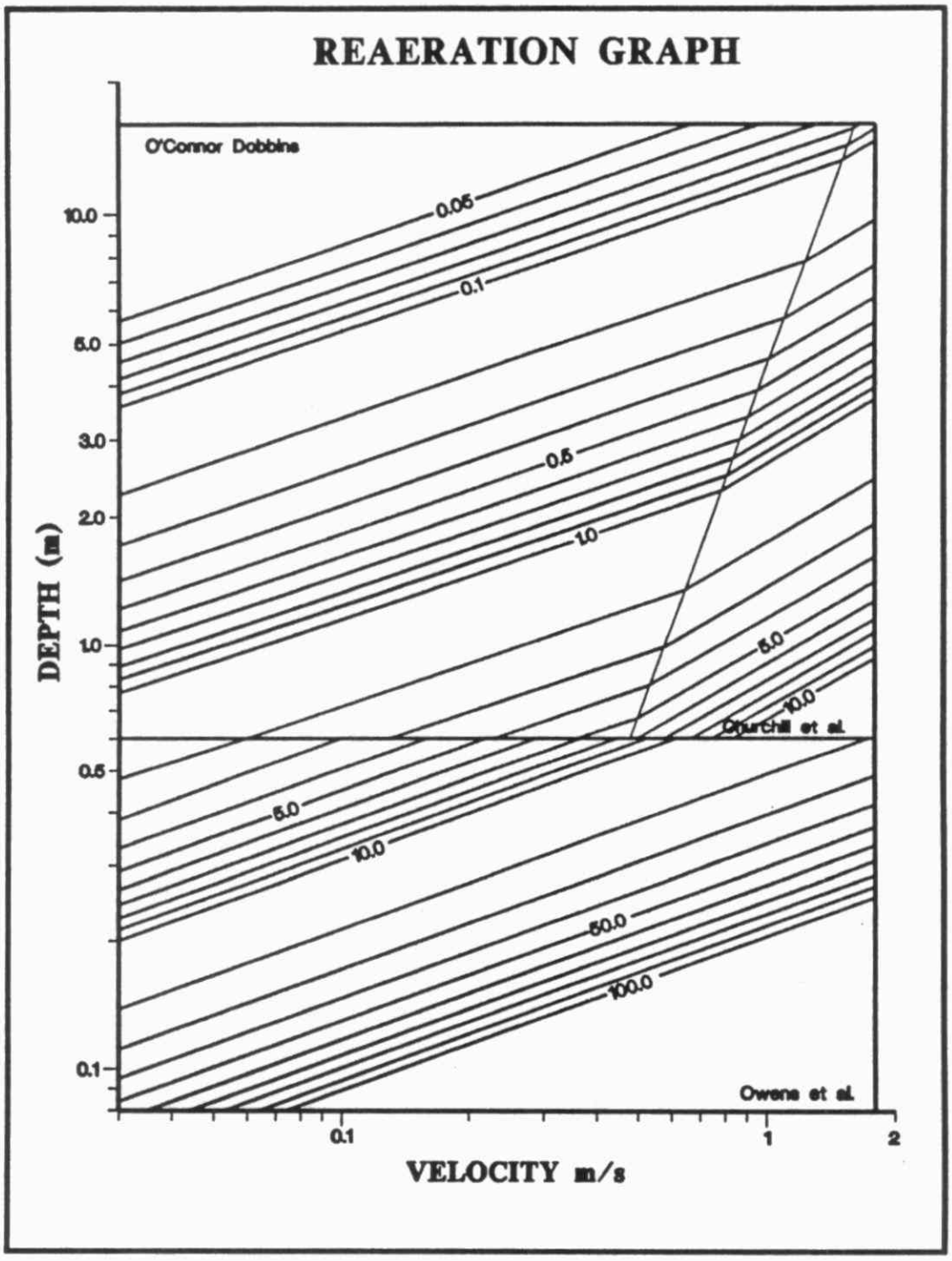

Figure 6-2 Reaeration Coefficient (20°C) Variation with Depth and Velocity

Thomann and Mueller (1987) also generally recommend the O'Connor-Dobbins relation although they note that it tends to underestimate the coefficient for small streams. Examining Fig. 6-2, it appears that a range of 0.8 - 4.0 d^{-1} (at 20°C) is reasonable for streams in the South Nation watershed.

6.2.2 *The BOD Exertion Coefficient*

The BOD exertion coefficient is not as variable as the reaeration coefficient, but it is much more subject to stream conditions such as acclimation of microorganisms to the waste. There are a few empirical expressions put forward for BOD exertion (Tetra Tech, 1978 and Thomann and Mueller, 1987) but there is little rationale behind them. However, Tetra Tech (1978) included an extensive survey of BOD exertion constants measured in situ. From a survey of these data, a range of 0.05 - 0.50 d^{-1} included more than 95% of the reported values. Reported temperatures were generally around 20°C for these coefficients. The range of 0.05 - 0.50 d^{-1} at 20°C was chosen for the BOD exertion coefficient.

BOD decay in any reach is calculated according to the usual first-order relation:

$$\frac{dL}{dt} = -k_1 L \qquad (6\text{-}4a) \qquad L_f = L_o e^{-k_1 t} \qquad (6\text{-}4b)$$

where L_o and L_f are initial and final ultimate BOD concentrations, respectively, in the reach

Laboratory measured values for BOD were reported on a $BOD_{5,20}$ (5 day BOD at 20°C) basis. Laboratory determined $BOD_{5,20}$ concentrations were converted to ultimate BOD (BOD_u) values for input into the models with Eq. 6-5. The commonly assumed value of 0.23 d^{-1} (base e) was used for the laboratory rate constant (Brown and Barnwell, 1987). This applied to lagoon effluent BOD_5 and measured river BOD_5 concentrations.

$$L_5 = L_u\left(1 - e^{-k_L(5)}\right) \qquad (6\text{-}5)$$

where L_5 is BOD_5
L_u is BOD_u
k_L is the BOD rate coefficient for laboratory conditions

6.2.3 *The Ammonia Oxidation Coefficient*

Temperature and pH have a more pronounced effect on the rate of nitrification compared to other phenomena (Tetra Tech, 1978). The majority of reported values for k_N in Tetra Tech (1978) were in the range of 0.1 - 0.54 d^{-1} at temperatures ranging from 20 to 28°C. The QUAL2E manual (Brown and Barnwell, 1987) reports a typical range of 0.1 - 1.0 d^{-1} for the rate coefficient for hydrolysis of ammonia. Thomann and Mueller (1987) report that values of nitrogenous BOD exertion (nitrogenous oxygen demand) coefficients are typically in the range of 0.1 - 0.5 d^{-1} at 20°C. The range of 0.1 - 1.0 d^{-1} at 20°C was selected.

The rate of ammonia decay can also be separately calculated according to a first-order relation:

$$\frac{dN}{dt} = -k_N N \qquad (6\text{-}6a) \qquad N_f = N_o e^{-k_N t} \qquad (6\text{-}6b)$$

where N_o and N_f are the initial and final ultimate NH_{3T} concentrations, respectively, in the reach

6.2.4 *Sediment Oxygen Demand*

SOD is the most highly variable of all phenomena considered. The usual environmental factors of pH, temperature and nutrient availability influence sediment decay. Bottom sediments are more remote from the supply of oxygen and different flora are at work in the benthos. Both aerobic and anaerobic degradation of organic sediments occur. The distance of a stream location from a waste source has a significant impact on the amount of degradable sediment being deposited and the rate of oxygen demand due to sediment. Chemical composition of sediment is another factor, particularly with respect to metals which may exhibit inhibitory or toxic reactions.

There is no universal agreement on factors responsible for SOD and no general acceptance of a single expression for calculating rate of SOD. Numerical values for SOD presented in Tetra Tech (1978) range from 0.1 - 10.0 g/m²/d, a variation of two orders of magnitude. For streams that are not exposed to a continuous wastewater effluent discharge, 5 g/m²/d is a conservative upper limit.

The areal rate of SOD must be related to the volume of the water column to determine the volumetric rate of SOD as follows:

$$s = \frac{k_s}{d} \qquad (6\text{-}7)$$

where k_s is areal rate of SOD
d is depth of the stream

Depth of the South Nation River generally varies from 0.6 to 1.3 m as noted in Sec. 6.2.1. Using a range of 0.1 - 5.0 g/m²/d for k_s and this depth range, s varies from 0.07 to 8 mg/L/d.

The QUAL2E manual (Brown and Barnwell, 1987) gives a default value of 1.06 for Θ. Tetra Tech (1978) reports that a Θ value of 1.072 is accepted by a large number of researchers. The latter value was chosen for use in this study.

6.2.5 *Coefficient Summary and Variation with Temperature*

Except for SOD, default values for Θ values given in the QUAL2E manual ((Brown and Barnwell, 1987) were assumed to be valid. These values are typical of those found in textbooks and reported in references noted above. The values are given in Table 6-1.

For the modelling runs, three sets of coefficients were considered: one with the values of the coefficients that produces the least stress on the receiving water; one at median values of the coefficients; and, one at coefficient values that produce the most severe conditions. The coefficient sets are also included in Table 6-1.

6.3 *OTHER MODELLING CONDITIONS*

Maximum temperatures result in the most severe DO uptake conditions. The maximum temperatures (based on 75th percentile values) for the windows of interest are given in Appendix H. The inflow waters entering the river between points of confluence or effluent discharge must be characterized for DO, BOD_u and NH_{3T}. The background BOD_5 and NH_{3T} of the inflow water were established at 2.3 and 0.18 mg/L, respectively in Chapter 4. With a rate constant of 0.23 d^{-1}, a BOD_5 of 2.3 mg/L is equivalent to a BOD_u of 3.4 mg/L.

Table 6-1 Coefficient Values[1]

Coefficient (d^{-1})	Range	Set 1 Value (d^{-1})	Median Set (Set 2) Value (d^{-1})	Set 3 Value (d^{-1})	Θ
k_1	0.05 - 0.50	0.05	0.25	0.50	1.047
k_2	0.8 - 4.0	4.0	2.0	0.8	1.024
k_N	0.1 - 1.0	0.1	0.5	1.0	1.083
s^2	0.07 - 8.0	0.02	2.5	5	1.072

1. All coefficients are given at 20°C.
2. SOD was assumed to only occur in fall.

The background DO of inflow waters was established by examining the 25th percentile DO values for spring and fall seasonal data at stream quality monitoring stations (Appendix E). A value of 8.0 mg/L is at or slightly below most of the 25th percentile values except for Chesterville. The extremely low 25th percentile value of 3.4 mg/L at Chesterville for the spring is not considered to be representative compared to the bulk of the data. Therefore the value of 8.0 mg/L was chosen for inflow DO. Using this approach to gauge the DO of inflow waters is conservative since stream DO is more highly influenced by oxygen consuming phenomena than reaeration phenomena.

The DO of effluents from all processes was taken to be 4.0 mg/L. For lagoons that discharge in spring, as noted above, their effluent must pass through a post aeration cell to strip H_2S. This will increase DO concentrations in the spring lagoon effluent particularly for discharge immediately after ice cover has gone when the lagoon contents may have a very low or negligible DO. Lagoons are facultative and fall effluent will contain DO. The other processes are aerobic and their effluents are expected to contain measurable amounts of DO. The modelling exercise is not particularly sensitive to effluent DO concentrations.

6.4 THE MODELS

A network of interconnected spreadsheets supported by a number of custom designed macros was used to characterize the system and to simulate the unsteady state of the problem. The spreadsheets solved Eq. 6-2 between each node, and also found the minimum DO between each node. The initial condition for the furthest upstream node was the river flow with the background concentrations given above. Waste flows were input at the appropriate nodes as desired. The spreadsheet calculated the waste flowrate by dividing the total waste volume to be discharged by the time span of the window.

Seven different streams were set up in each spreadsheet: one each for the Bear Brook, Castor, Hess Creek, Moose Creek, Paxton Creek, Scotch and South Nation River systems. These streams all have point sources potentially discharging into them in the future. For the present condition, waste volumes of those point sources discharging in the future (Bourget, Crysler, Fournier, Moose Creek, St. Albert and St. Albert Cheese) were set to zero.

At each node where confluence occurred, the appropriate tributary flowrate was input. There was not confluence of a tributary at all nodes but the flow at each node was always increased relative to the flow at the immediate upstream node because there was an increase in drainage area. (Refer to Fig. 3-7 for the drainage areas at all nodes). All flow augmentations at a node were mixed with river flow at the final conditions for the upstream reach immediately above the node.

If there was more than one point source at a node (e.g., Chesterville where both the municipality and Nestle discharge), the second source was arbitrarily located at a dummy node that was located 0.1 km below the actual node. This allowed for separating the two sources and conveniently turning one or the other off or on as desired without compromising the accuracy of the solution. The velocity over the 0.1 km distance was assumed to be the velocity in the previous reach.

All waste flows were cumulatively added to the base flow as the solution progressed downstream. The amounts of BOD and NH_{3T} decay (Eqs. 6-4a - 4b and 6-6a - 6b, respectively) were computed in each reach as well as the change in the DO deficit. All rate constants were adjusted to the specified temperature which was assumed to be constant for all streams.

A brief step-by-step explanation of the fundamental equations in the spreadsheet follows using the first two nodes (Q and P) for the South Nation River at spring 7Q20 maximum window flow conditions. Lagoon treatment will be used for purposes of explanation.

1. The river flow coming into Spencerville is 0.391 m^3/s. The present condition volume of waste to be discharged is 51,100 m^3 and the maximum window duration is 53 days. Therefore Q_w equals 0.0112 m^3/s. The river at the background concentrations given above was mixed with waste flowrate using effluent concentrations of BOD_u, NH_{3T} and DO for a lagoon. This determined the initial concentrations, L_o, N_o and D_o (4.49, 0.565 and 2.71 mg/L respectively).

2. The travel time between nodes Q and P is 16.35 - 13.87 = 2.48 days. This time was adjusted by taking the ratio of $(.391/.402)^{0.4}$ according to Eq. (3-3). The new travel time in the reach is 2.45 days.

3. Now the final deficit in the reach Q-P immediately before point P was calculated using Eq. 6-2. The final BOD_u and NH_{3T} concentrations immediately before point P were calculated (according to Eqs. 6-4b and 6-6b, respectively) to be 1.85 and 0.139 mg/L respectively. The spreadsheet also computed the minimum DO percent saturation between Q and P to be 71.4%.

4. The final values of the deficit, BOD and NH_{3T} calculated in (3) and present in the total flow of 0.402 m^3/s were now mixed with the additional flow coming in at P to determine the initial concentrations at P. The additional flow added at P was assumed to be at inflow background concentrations. The flow at P includes the flow calculated from the drainage area correlation plus the flow from all upstream sources which in this case is only the waste flow from Spencerville.

The process was then repeated for the reach P to O and further nodes upstream adding in waste flows as required. Thus for each node, initial and final values of the deficit, ammonia and BOD are reported. Initial values are the concentrations at the node and final values are the concentrations immediately before the next downstream node. All spreadsheets are given in Appendix G. Appendix G(a) contains spreadsheets results with Coefficient Set 3, which is the limiting case for DO, and Appendix G(b) contains spreadsheets results with Coefficient Set 3 and $K_N = 0.1\ d^{-1}$, which is the critical case for ammonia.

The spreadsheet results have been checked with QUAL2E results and they were found to be in very close agreement. This is not surprising since the finite difference solution (implemented in QUAL2E) of Eq. 6-1 should result in the closed form solution of this equation with some deviation due to numerical dispersion.

6.4.1 General Solution Procedure

The general approach to staging discharges has been outlined in Sec. 6.1.2. Separate spreadsheets were setup for the maximum, average and minimum windows in the spring with the corresponding flowrates and travel times for each window. For the fall similar spreadsheets were created. However, as discussed in Sec. 6.1.2, these were not used to find solutions. They did indicate that PWQO cannot be met in most locations even if there are no discharges.

The first issue to be examined was the feasibility of discharging effluent from the remote communities as close as possible to the communities' locations. Communities identified in Chapter 5 as having a problem with local discharge were studied. With lagoon treatment, the discharge point was moved downstream to a distance where sufficient dilution capacity was available. If the distance from the community to the required discharge point in the local stream was larger than the shortest distance of the community from the South Nation River, effluent was re-routed to the South Nation.

The above exercise was repeated with different types of treatment installed. The extended aeration and New Hamburg processes produce effluent with NH_{3T} concentrations of 5 and 4 mg/L, respectively at all times during the year (Table 2-9). If a confluence was reached as a waste source was gradually relocated further downstream, the instantaneous increase in flow (and dilution capacity) could reduce the need for upgraded treatment. Particularly for small streams the increase in flow incurred travelling downstream is very small compared to the increase in flow at a node where tributary inflow occurs. Therefore as a general policy when dilution capacity was found to be sufficient at a point in a reach between nodes, the discharge point was located at the nearest downstream node where a confluence occurred. This procedure provides a safety factor that sufficient dilution capacity is available. Flow estimation in the smaller streams is less accurate than for the gauged South Nation River which is another reason for this safety factor.

This established the discharge locations for many of the remote communities. The staging exercise as outlined in Sec. 6.1.2 was then followed. Lagoon treatment was assumed for all communities for the initial optimization. Industries were assumed to have effluent quality as discussed in Sec. 2.7. All fall windows were examined, but it was found that only a few communities could discharge an insignificant amount of waste without violating any PWQOs.

The maximum, average and minimum spring windows were optimized sequentially. A point source was only allowed to discharge in a window if it could discharge its total waste volume in that window without violating the PWQO. Discharge rates found in the maximum window spreadsheet were automatically input (or *linked*) in the average window spreadsheet as base conditions. Discharge rates found in the maximum and average window spreadsheets were linked into the minimum window spreadsheet and the remaining point sources were discharged.

All waste volumes not discharged during the maximum and average windows must be discharged during the minimum window. After discharging all remaining wastes in the minimum window, the stream NH_3 profile was examined. At points of violation of PWQO, some waste sources at or upstream of the node need to be upgraded. The second round optimization was conducted with extended aeration installed for sources causing PWQO violations. The approach was similar to that outlined above. Further rounds of optimization involved installing extended aeration processes and/or relocating discharge location further downstream. Most iterative tasks were automated by custom-designed macros.

The DO profiles were also examined, but it was found that in the spring DO was not the limiting factor.

Additional comments on relocation of discharge points and advanced treatment requirements will be provided as results are presented. But one comment on advanced treatment is warranted at the outset. It has been noted that this study was confined to an examination of spring and fall discharges only. The advanced treatments produce effluent of higher quality on a year-round basis and in some cases it will undoubtedly be possible to discharge a portion of the annual waste volume outside of the spring and fall periods.

This will have the beneficial effect of reducing the volume of wastes being discharged into the South Nation system during the highly utilized spring period. This may relax treatment upgrades for some other communities and will reduce the distance required to relocate some effluent discharge points. The issue will be revisited in the discussion.

6.5 MODELLING RESULTS

The model was run with k_N of 0.1 d^{-1} which is the critical condition for ammonia, and with k_N of 1.0 d^{-1}, which is the critical condition for DO. All rate coefficient values will be addressed at their 20°C value. The spreadsheet adjusted coefficients to appropriate values at the specified temperature for the window, and automatically selected the coefficients which yielded the most negative effect on the pollutant being examined (ammonia or BOD).

6.5.1 30Q20 Streamflows Solution

6.5.1.1 The Optimum Present 30Q20 Solution

The following point sources would have to relocate their effluent discharge points or upgrade their treatment processes: Ault, Maxville, Williamsburg, and Winchester. A discussion of each source follows.

Ault

Without Winchester discharging into the Castor system Ault would have to discharge its effluent further downstream in the Castor system. From analysis of dilution requirements and comparing Figs. 1-1 and 3-7, the nearest feasible discharge point would be very near node AB in Fig. 3-7. (See Winchester below for further discussion of Ault).

Maxville

For lagoon treatment, Maxville would have to discharge its effluent at node T in the Scotch River system. The confluence of the West Branch of the Scotch River with the Cumming Drain produces a significant rise in dilution capabilities at this node.

Williamsburg

Williamsburg would have to upgrade its treatment to extended aeration or better treatment. Another possibility for this community would be to pipe its effluent to the South Nation or St-Lawrence Rivers.

Winchester

The load discharged by Winchester is much higher than the load discharged by Ault due to the higher flows produced by Winchester and the lower effluent quality for Winchester when the community is treating its waste with lagoons. Scenarios were examined where both Ault and Winchester were discharging into the Castor system as well as only one of each of these sources discharging into the Castor system. With only Winchester discharging into the Castor system, Embrun and Russell would have to upgrade their treatment or relocate their effluent discharge points to the South Nation River. This was not the case with only Ault discharging into the Castor system. Therefore the least cost solution for the Castor system and the south Nation watershed is to relocate Winchester's discharge location to the South Nation (node N is the nearest point on the South Nation River).

The optimum wastewater allocation and treatment requirements for the present 30Q20 conditions are given in Table 6-2. Except for Williamsburg, lagoon treatment will suffice for all communities. Because of the small flow in the McMartin Drain, Williamsburg must upgrade its treatment to the extended aeration process in addition to relocating its discharge point. The NH_{3T} profiles for the maximum, average and minimum spring windows are given in Figs. 6-3, 6-4, and 6-5 respectively. The present condition under 30Q20 flows is the least severe situation. Except for relocation of some point sources discharging into small tributaries, there is no need to stagger discharges through the different windows and ammonia concentrations are well below the stream ammonia limit.

DO percent saturation and BOD profiles for the present condition are shown in Figs. 6-6 - 6-8. Coefficient Set 3 (Table 6-1) which is the worst case condition for DO was

used in producing Figs. 6-6 to 6-8. For Figs. 6-3 to 6-5, a K_N of 0.1 d^{-1} was used. This is the worst case for ammonia and the limiting condition.

It can be seen in Figs. 6-6 to 6-8 that in the spring, DO is always well above the PWQO minimum of 47% saturation. Ammonia was therefore the parameter to address for the spring months, especially in the low-flowing tributaries.

6.5.1.2 The Optimum 30Q20 Future Solution

All dischargers are able to release at the same locations within the watershed as in the 30Q20 present solution. However, it was necessary to change the discharge period of some sources to the average window in order to avoid severe stress on the receiving waters. Except for Williamsburg lagoon treatment was feasible for all point sources. The optimum wastewater allocation and treatment requirements for the future 30Q20 conditions are given in Table 6-3. The NH_{3T} profiles for the future maximum, average and minimum spring windows are given in Figs. 6-9, 6-10, and 6-11, respectively.

DO percent saturation and BOD profiles for the future condition are shown in Figs. 6-12 - 6-14. Coefficient Set 3 (Table 6-1) which is the worst case condition for DO was used in producing Figs. 6-12 to 6-14. For Figs. 6-9 to 6-11, a K_N of 0.1 d^{-1} was used. This is the worst case for ammonia.

As observed for the present conditions, DO is in general not problematic in spring months of the future solution.

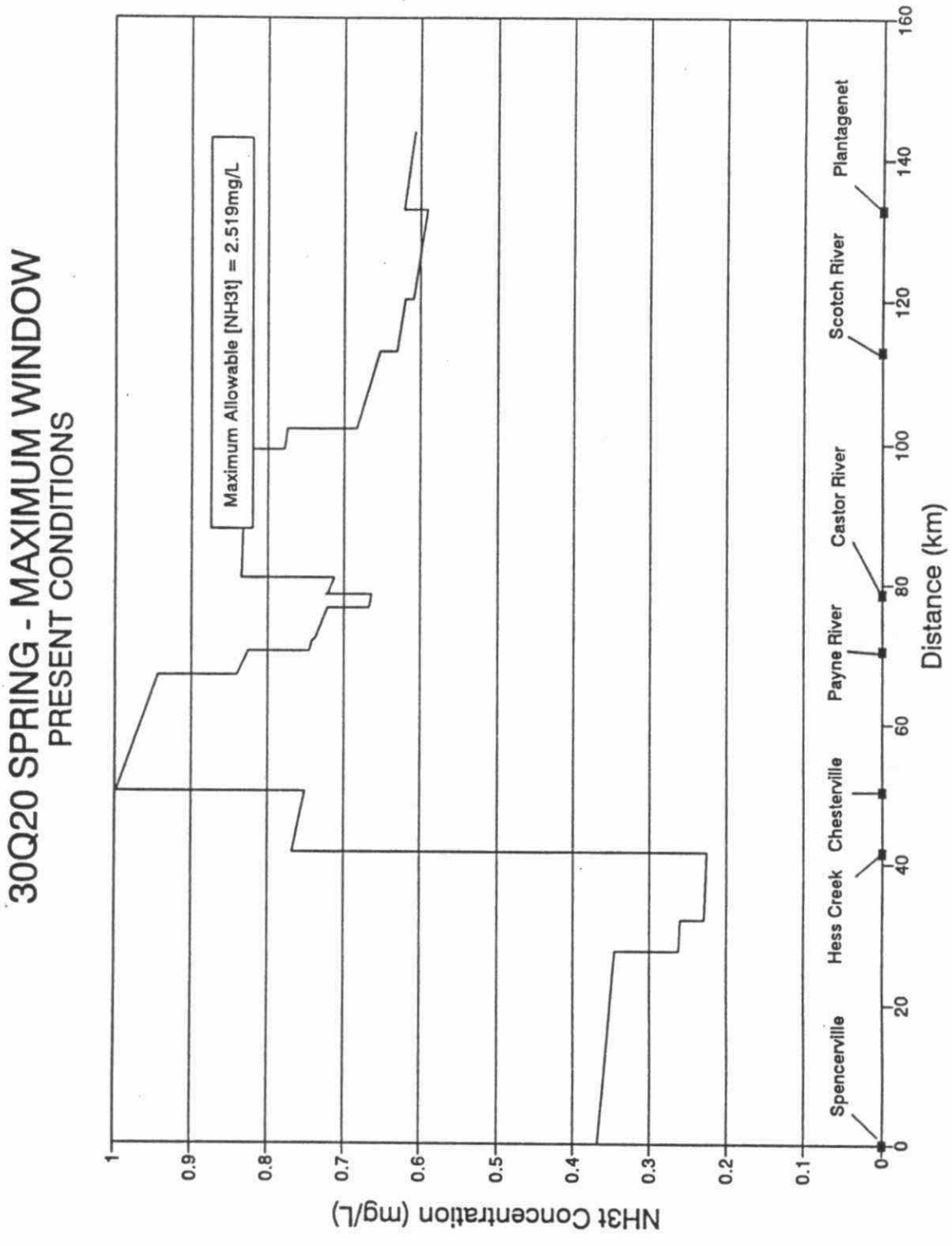

Figure 6-3 Spring 30Q20 Maximum Window NH_{3T} Profile for Present Conditions

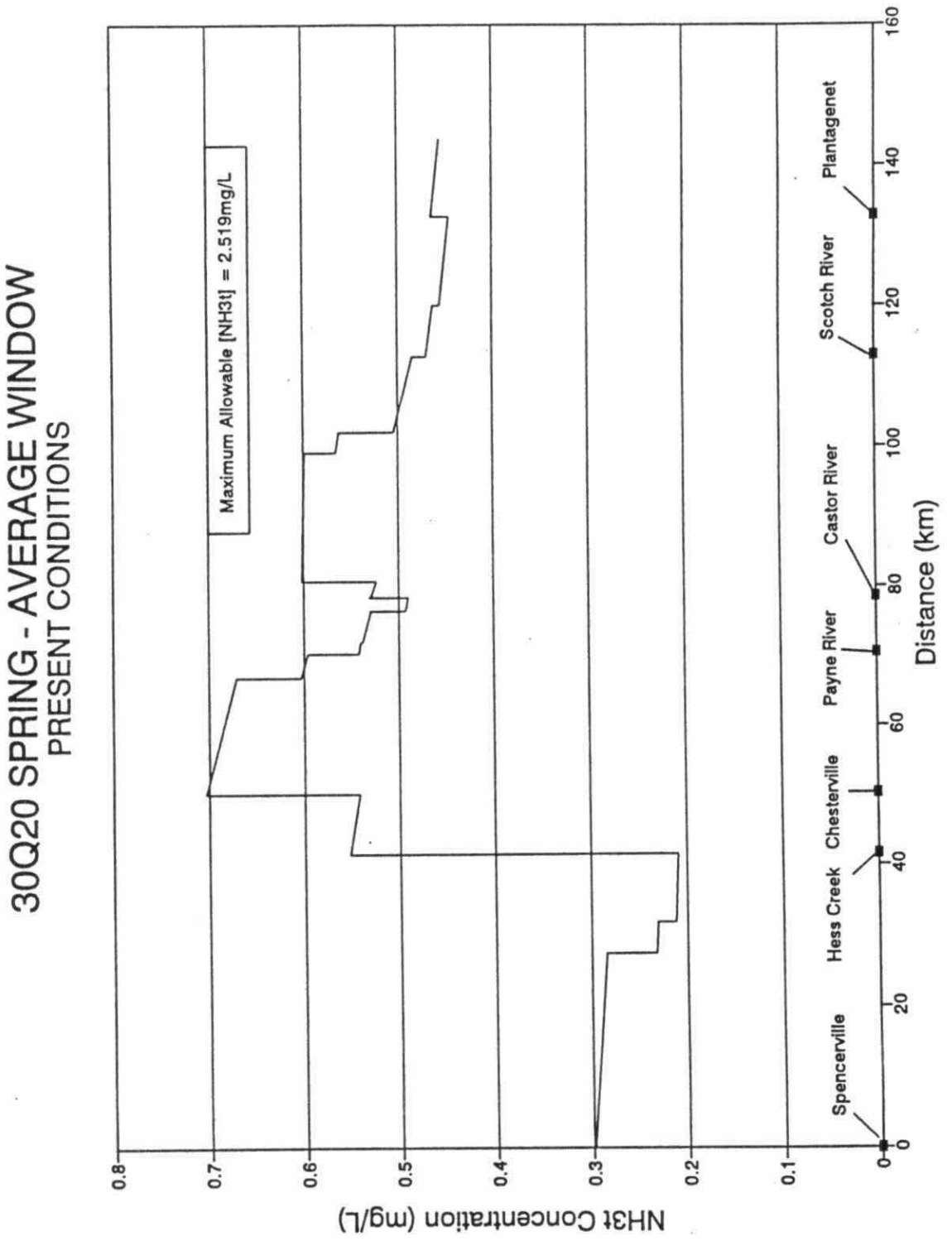

Figure 6-4 Spring 30Q20 Average Window NH_{3T} Profile for Present Conditions

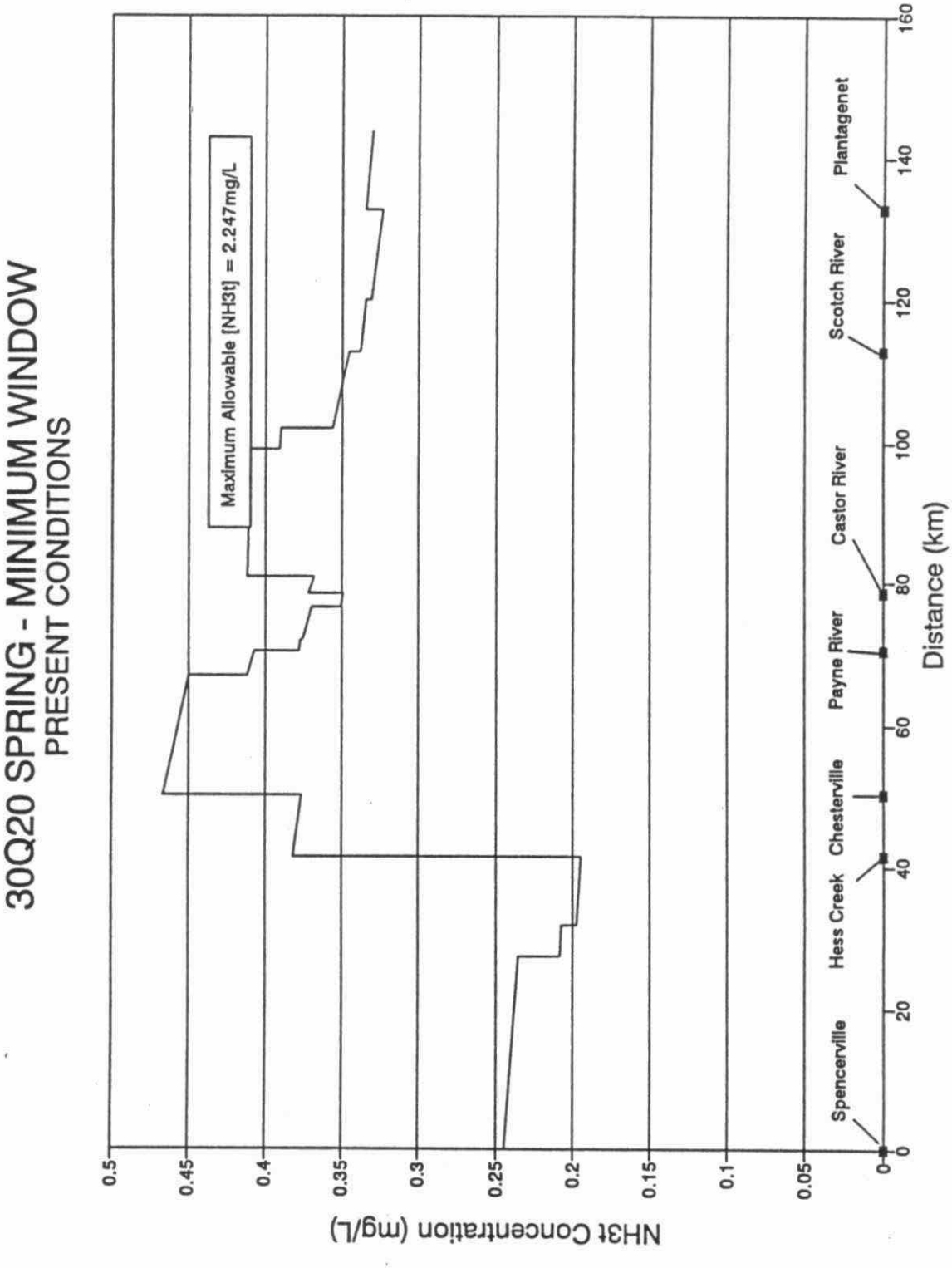

Figure 6-5 Spring 30Q20 Minimum Window NH_3 Profile for Present Conditions

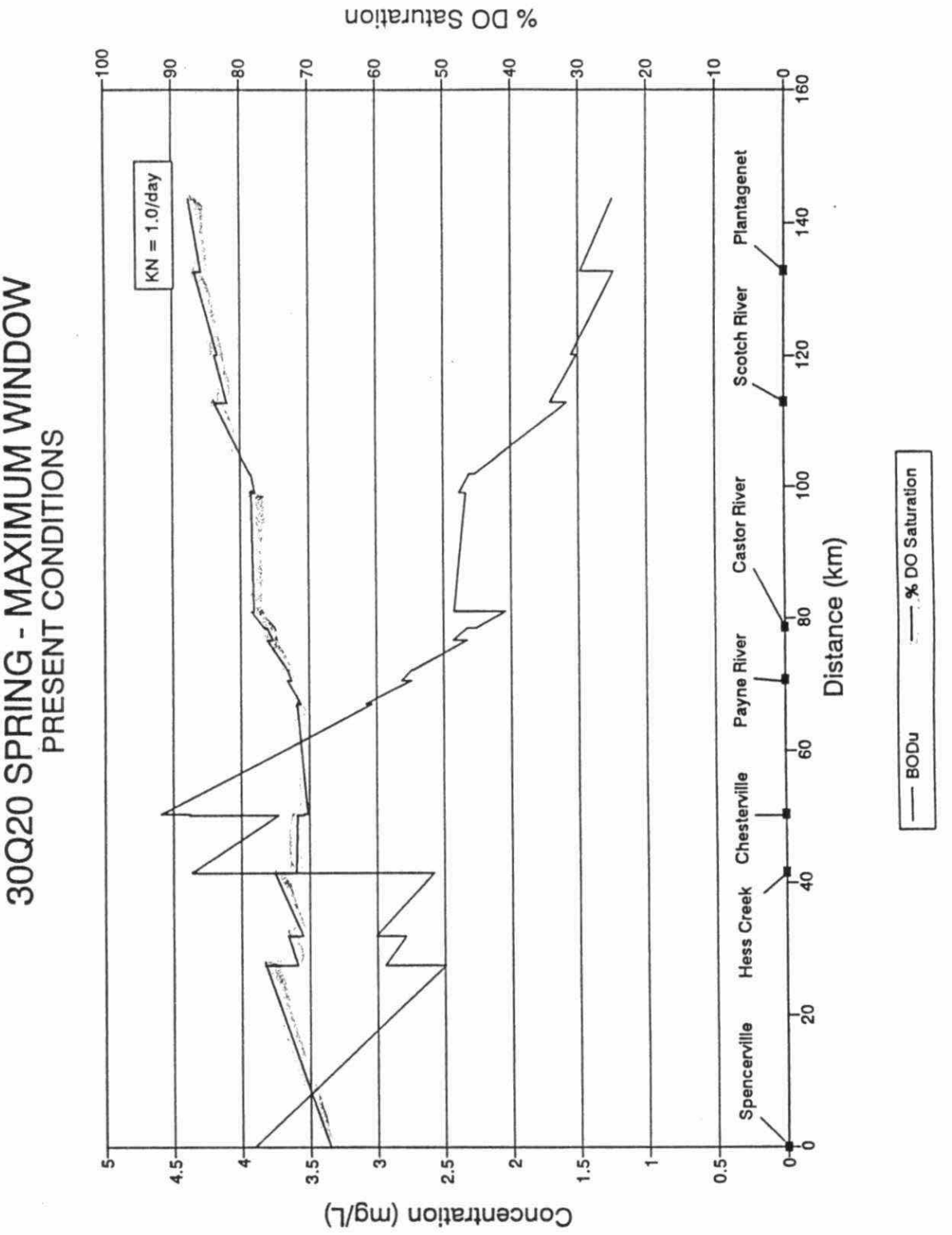

Figure 6-6 Spring 30Q20 Maximum Window DO and BOD Profiles using Coefficient Set 3

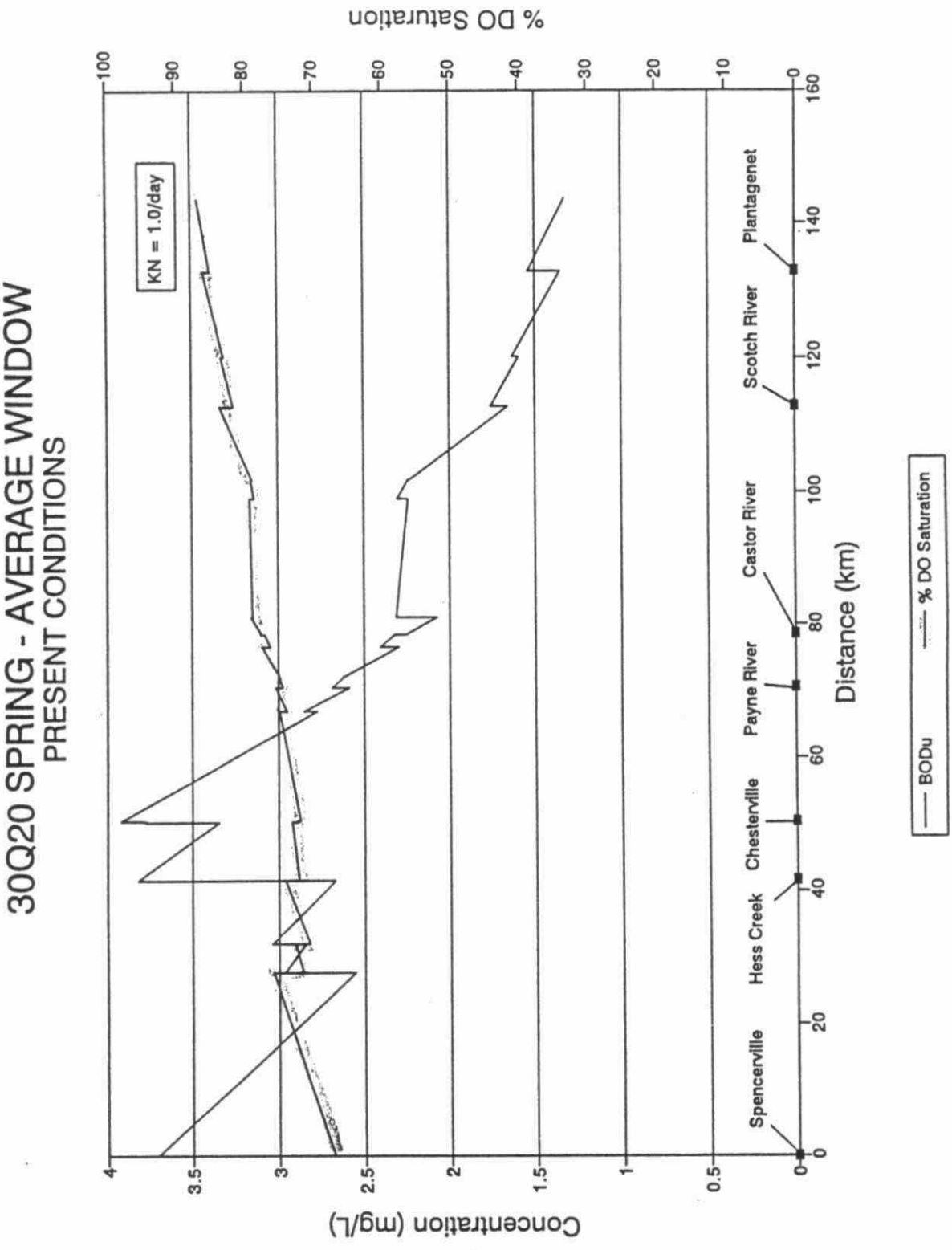

Figure 6-7 Spring 30Q20 Average Window DO and BOD Profiles using Coefficient Set 3

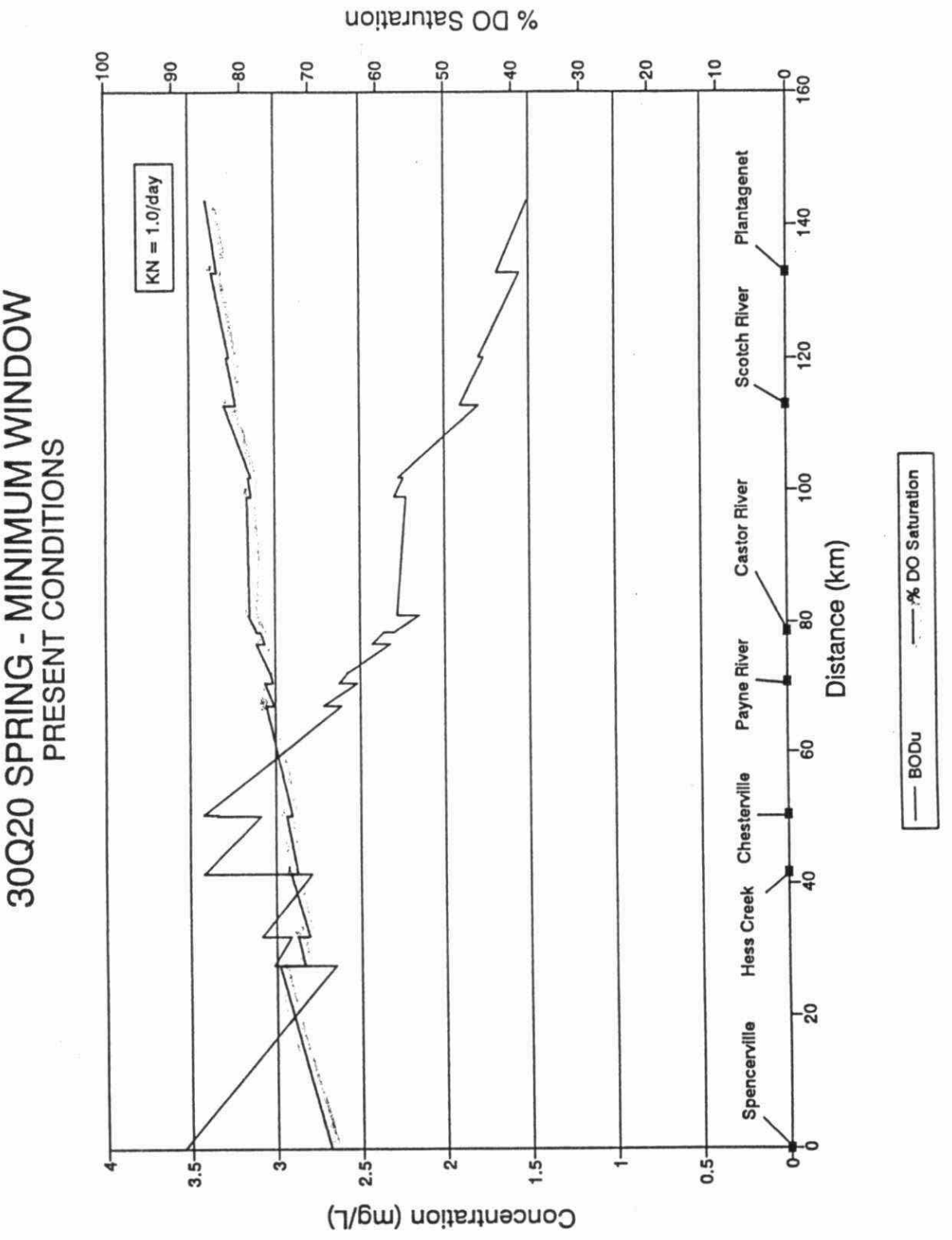

Figure 6-8 Spring 30Q20 Minimum Window DO and BOD Profiles using Coefficient Set 3

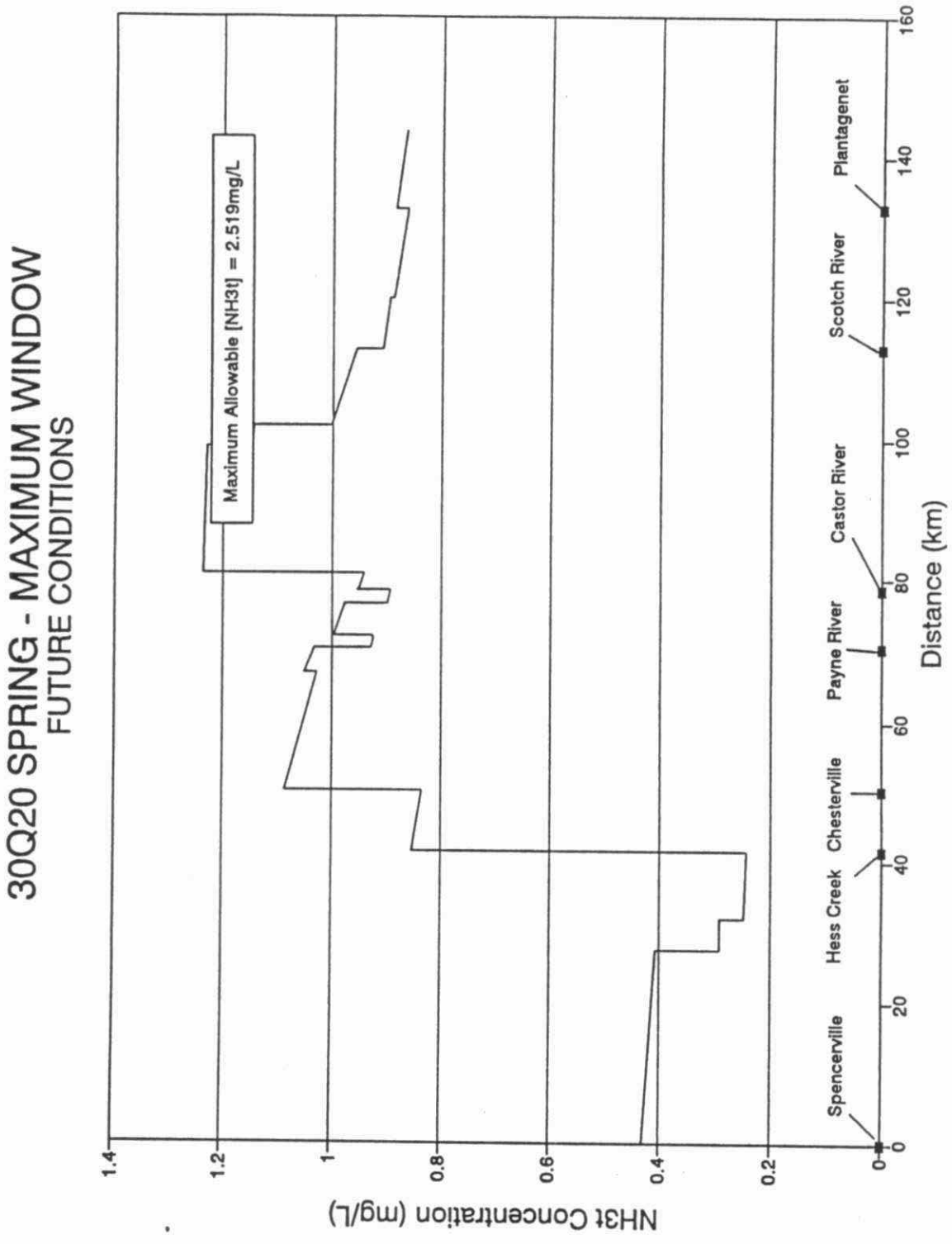

Figure 6-9 Spring 30Q20 Maximum Window NH_{3T} Profile for Future Conditions

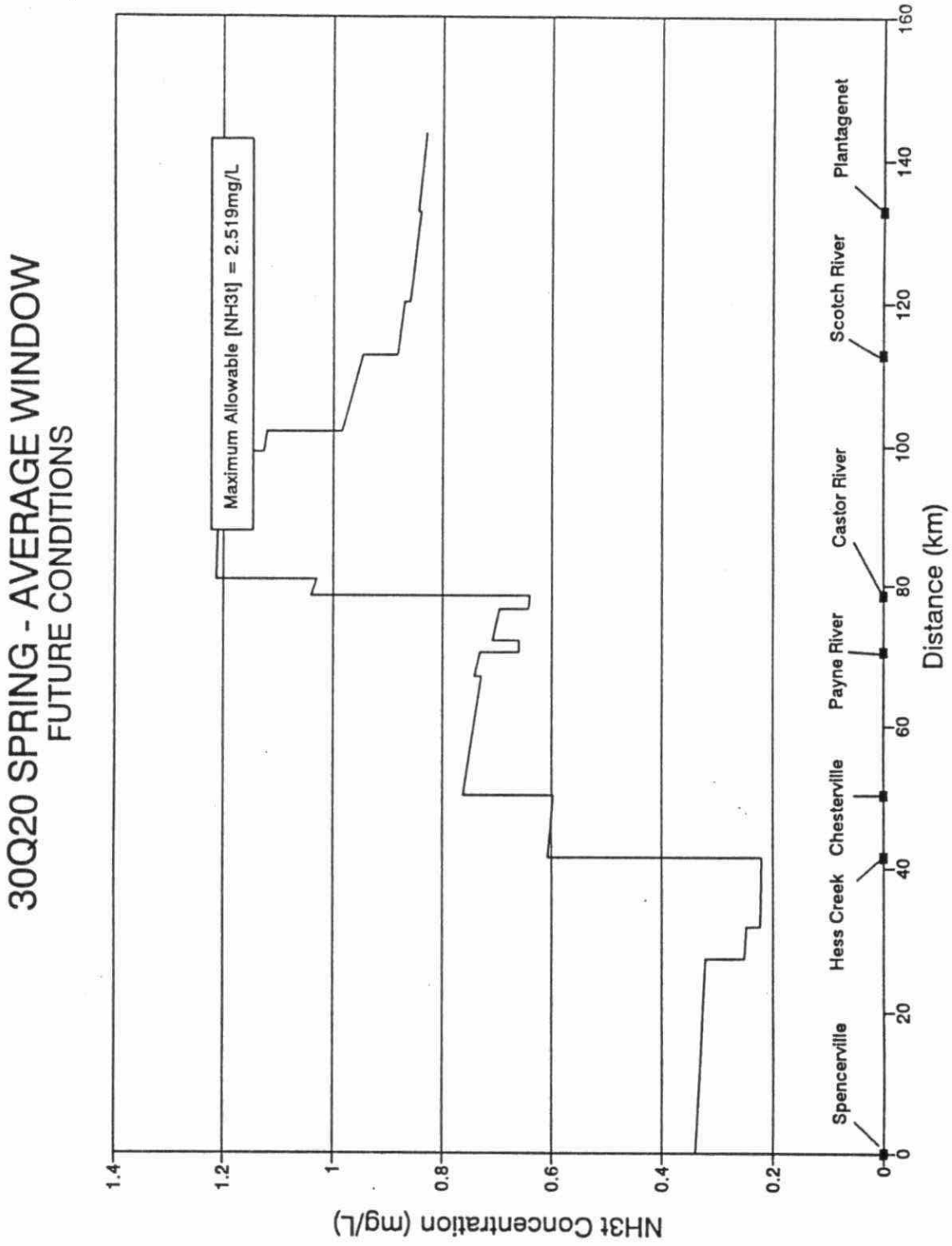

Figure 6-10 Spring 30Q20 Average Window NH_{3T} Profile for Future Conditions

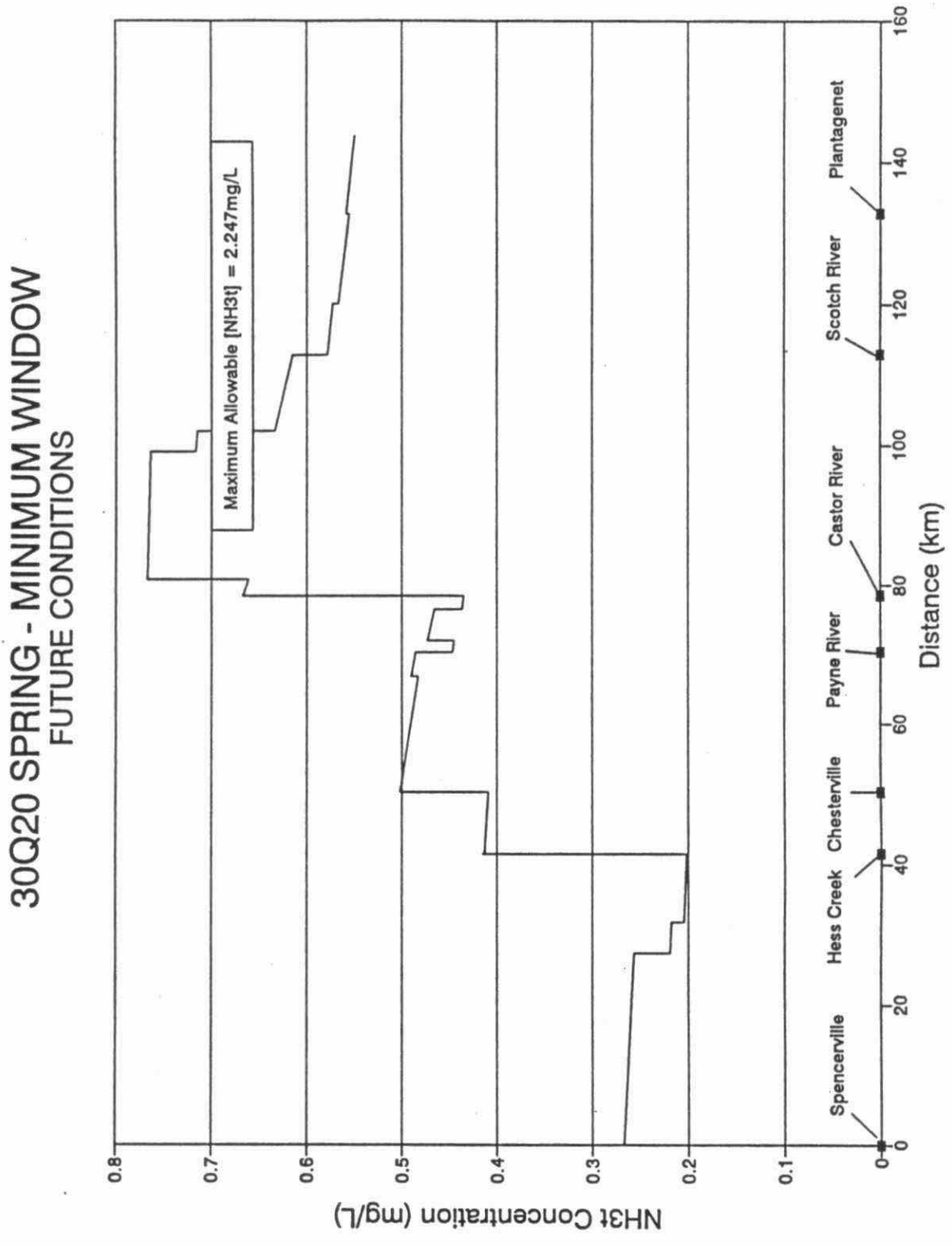

Figure 6-11 Spring 30Q20 Minimum Window NH_{3T} Profile for Future Conditions

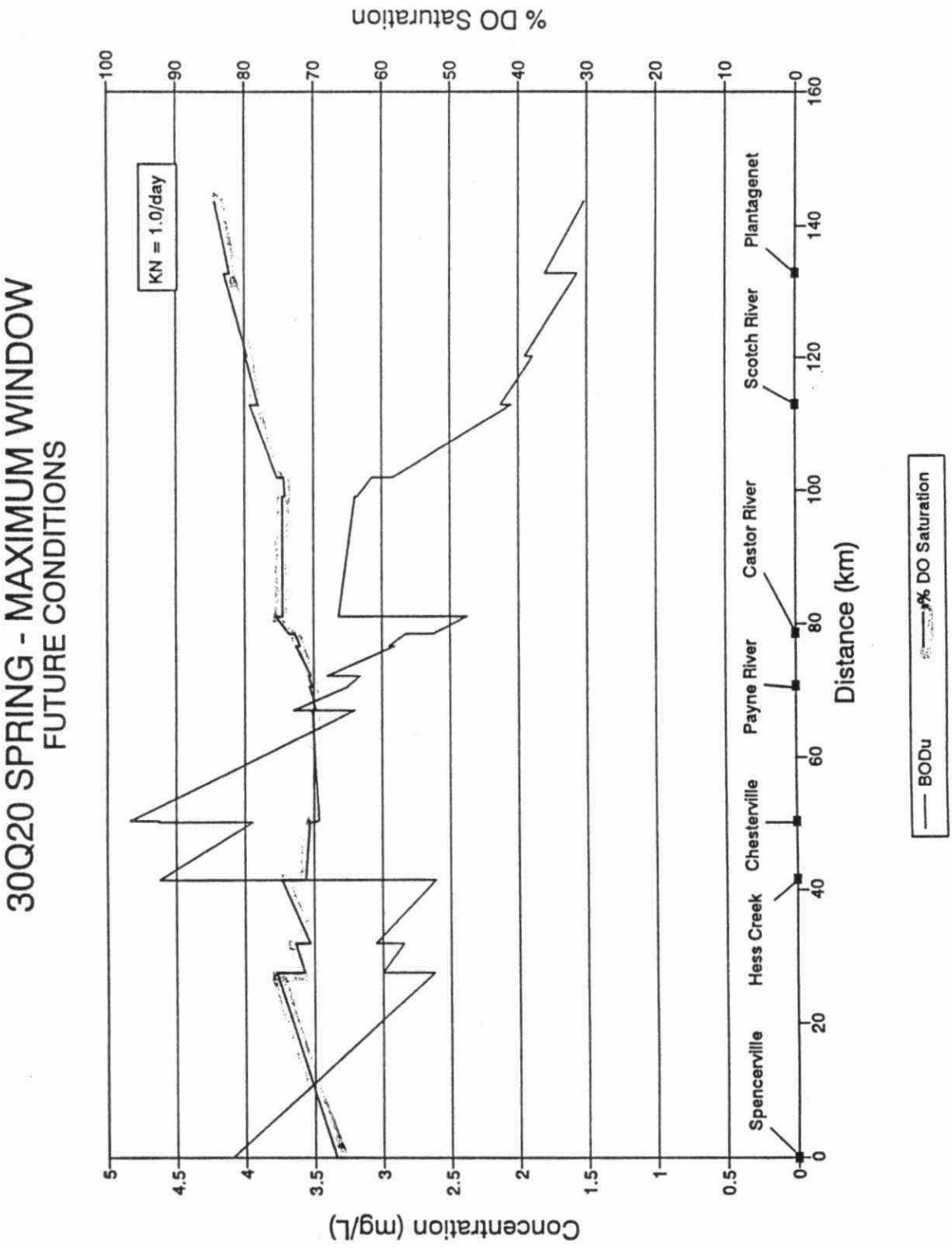

Figure 6-12 Spring 30Q20 Maximum Window DO and BOD Profiles for Future Conditions using Coefficient Set 3

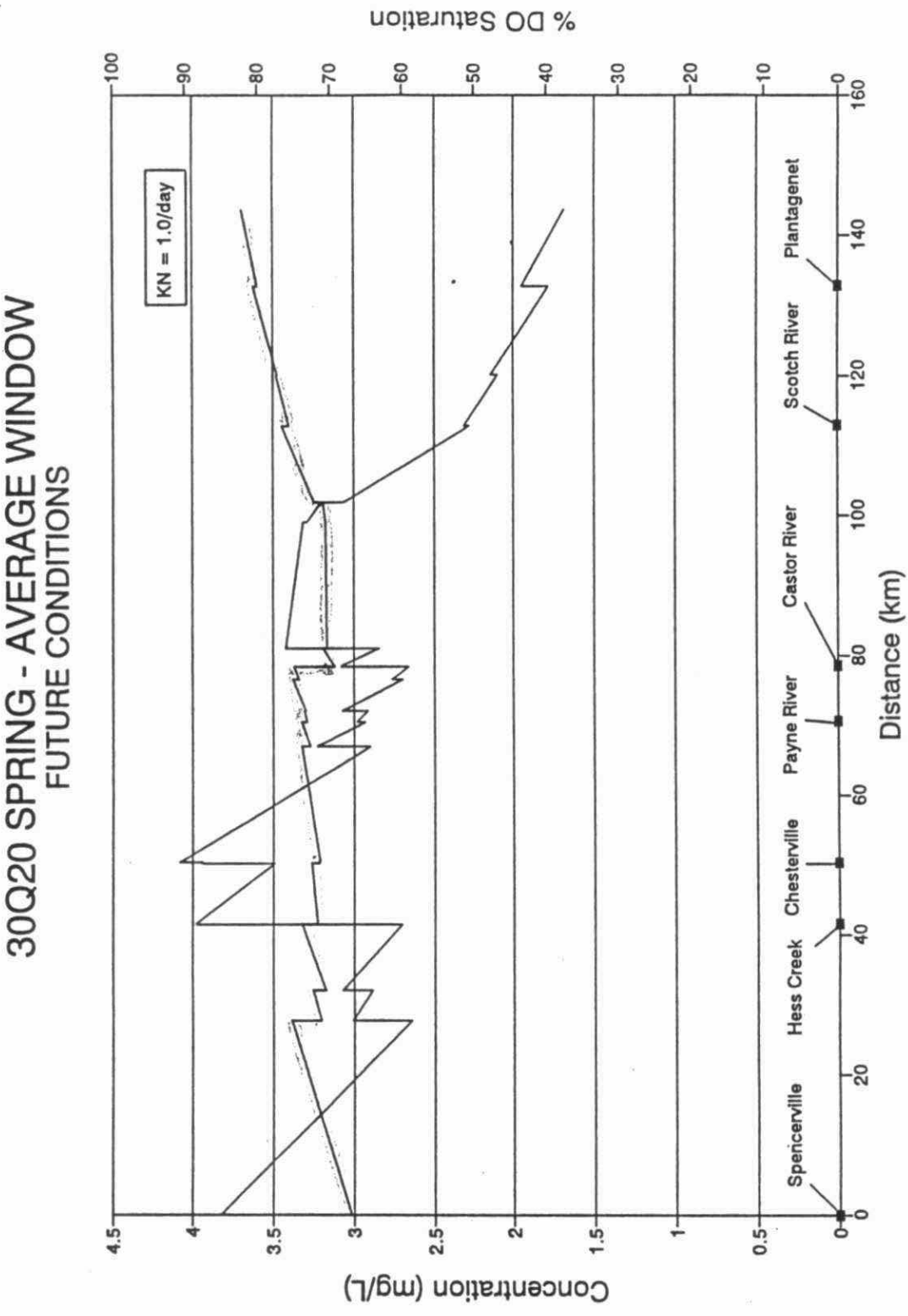

Figure 6-13 Future Condition Spring 30Q20 Average Window DO and BOD Profiles using Coefficient Set 3

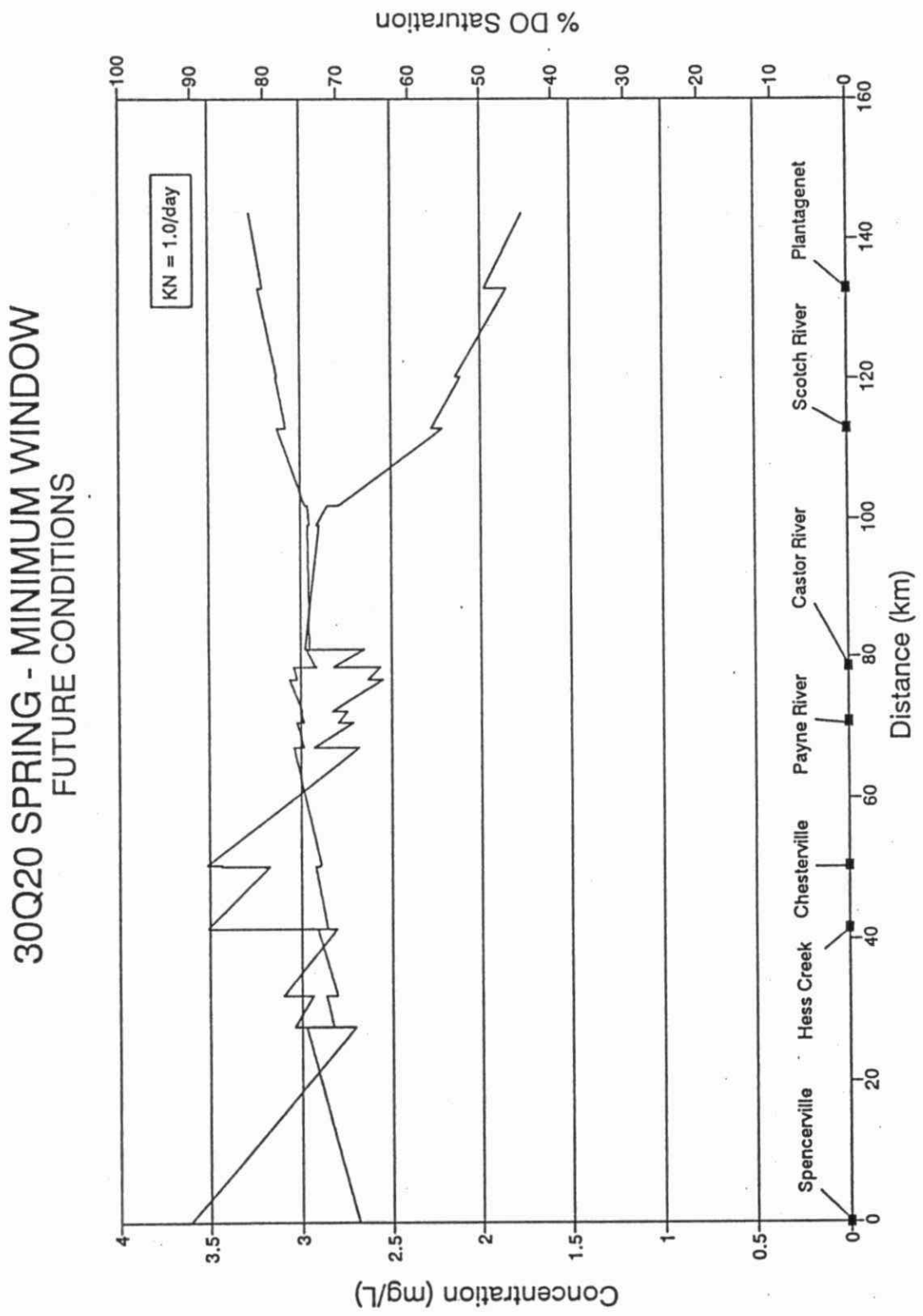

Figure 6-14 Future Condition Spring 30Q20 Minimum Window DO and BOD Profiles using Coefficient Set 3

Table 6-2 Optimum Wastewater Allocation and Treatment for Present 30Q20 Flows

Discharge Window	Point Source	Treatment[2]	Discharge Location[3]	Volume Discharged (m^3)	BOD_5 Loading (kg)	NH_{3T} Loading (kg)	Discharge Rate (m^3/s)
Max March 16 to April 24	Ault	I[1]	AB	193800	970	1180	0.0561
	Casselman	L	G	365000	10950	5110	0.1056
	Chesterville	L	M	296015	8880	4140	0.0857
	Embrun	L	Z	474500	14200	6640	0.1373
	Maxville	L	T	59860	1800	840	0.0173
	Nestle	I[1]	M	225862	3390	1380	0.0654
	Plantagenet	L	B	225202	6760	3150	0.0652
	Russell	L	AA	193085	5790	2710	0.0559
	Spencerville	L	Q	51100	1530	715	0.0148
	St. Isidore	L	S	127000	3810	1780	0.0368
	Williamsburg	EA	AG	64100	960	320	0.0185
	Winchester	L	N	629600	18900	8810	0.1822

1. Industrial treatments are discussed in Sec. 2.7.
2. EA - extended aeration; L - lagoon (refer to Sec 2.7 for effluent concentrations).
3. Discharge locations are nodes on Fig. 3-7.

Table 6-3 Optimum Wastewater Allocation and Treatment for Future 30Q20 Flows

Discharge Window	Point Source	Treatment[2]	Discharge Location[3]	Volume Discharged (m^3)	BOD_5 Loading (kg)	NH_{3T} Loading (kg)	Discharge Rate (m^3/s)
Max March 16 to April 24	Ault	I[1]	AB	455000	2280	2780	0.1317
	Bourget	L	V	230200	6910	3220	0.0666
	Casselman	L	G	944300	28300	13200	0.2732
	Chesterville	L	M	318800	9560	4470	0.0922
	Crysler	L	L	219000	6570	3070	0.0634
	Fournier	L	R	50900	1530	710	0.0147
	Maxville	L	T	61300	1840	860	0.0177
	Nestle	I[1]	M	225900	3390	1380	0.0654
	Plantagenet	L	B	254100	7620	3560	0.0735
	Russell	L	AA	865300	26000	12100	0.2504
	Spencerville	L	Q	69400	2080	970	0.0201
	St-Albert	L	J	138400	4150	1940	0.0400
	St-Albert Cheese	I[1]	J	22600	340	140	0.0065
	St-Isidore	L	S	176700	5300	2480	0.0511
	Williamsburg	EA	AG	70800	1060	350	0.0205
	Winchester	L	N	717200	21500	10000	0.2075
Ave March 20 to April 20	Embrun	L	Z	1551100	46500	21700	0.5610
	Moose Creek	L	X	101600	3050	1420	0.0368

1. Industrial treatments are discussed in Sec. 2.7.
2. L - lagoon; EA - extended aeration (refer to Sec. 2.7 for effluent concentrations).
3. Discharge location nodes are indicated on Fig. 3-7.

6.5.2 7Q20 Streamflows Solution

6.5.2.1 The Optimum Present 7Q20 Solution

With 7Q20 flows and present conditions the following point sources would have to relocate their effluent discharge points: Ault, Maxville and Winchester. The requirements to relocate the discharge points for all of these point sources are the same as for 30Q20 conditions. Williamsburg must upgrade to extended aeration treatment or better.

The optimum wastewater allocation and treatment requirements for the present 7Q20 conditions are given in Table 6-4. The NH_{3T} profiles for the maximum, average and minimum spring windows are given in Figs. 6-15, 6-16, and 6-17 respectively. As seen from the table and figures, it is necessary to utilize all windows in the discharge scheme and the stream limit for ammonia is approached in a few instances. This is due to the smaller 7Q20 flows compared to the 30Q20 flows. Again, except for Williamsburg, lagoon treatment remains feasible for all communities.

DO percent saturation and BOD profiles for the present condition are shown in Figs. 6-18 - 6-20. Coefficient Set 3 (Table 6-1) which is the worst case condition for DO was used in producing Figs. 6-6 to 6-8. For Figs. 6-15 to 6-17, a K_N of 0.1 d^{-1} was used. This is the worst case for ammonia.

It can be seen in Figs. 6-18 to 6-20 that in the spring, DO is always well above than the PWQO minimum of 47% saturation. Ammonia remained the parameter to address for the spring months under 7Q20 flows.

6.5.2.2 The Optimum 7Q20 Future Solution

All dischargers are able to release at the same locations within the watershed as in the 7Q20 present condition solution. However, it is necessary to upgrade treatment at Embrun and Moose Creek to the extended aeration process. Releases occur in all three windows in order to avoid severe stress on the receiving waters.

Embrun

Embrun would have to upgrade to the extended aeration process or better. Both Embrun and Russell have very ambitious growth plans, and the Castor system will not be able to assimilate the additional waste from both municipalities with lagoon treatment in place at both communities. The total pollutant loading of these two municipalities must be reduced. Optimizations were conducted with extended aeration treatment for one of these communities and lagoon treatment for the other. It was found that Embrun must install extended aeration treatment to avoid relocating its discharge location. It is feasible to retain lagoon treatment for Embrun and relocate its discharge point to the South Nation River (Russell would have lagoon treatment). The cost analysis presented in Chapter 8 will dictate the choice between extended aeration with no discharge relocation or lagoon treatment with relocation of the discharge point to the South Nation River.

Moose Creek

Moose Creek discharges into a relatively small receiving stream. Under 7Q20 flow conditions, lagoon treatment will not suffice. Extended aeration (or better) is required to keep the receiving stream's water quality within PWQO.

The optimum wastewater allocation and treatment requirements for the future 7Q20 conditions are given in Table 6-5. The NH_{3T} profiles for the future maximum, average and minimum spring windows are given in Figs. 6-21, 6-22, and 6-23 respectively.

DO percent saturation and BOD profiles for the future condition are shown in Figs. 6-24 - 6-26. Coefficient Set 3 (table 6-1) which is the worst case condition for DO was used in producing Figs. 6-24 to 6-26. For Figs. 6-21 to 6-23, a K_N of 0.1 d^{-1} was used. This is the worst case for ammonia.

As observed for the present conditions, DO is in general not problematic in spring months of the future 7Q20 solution.

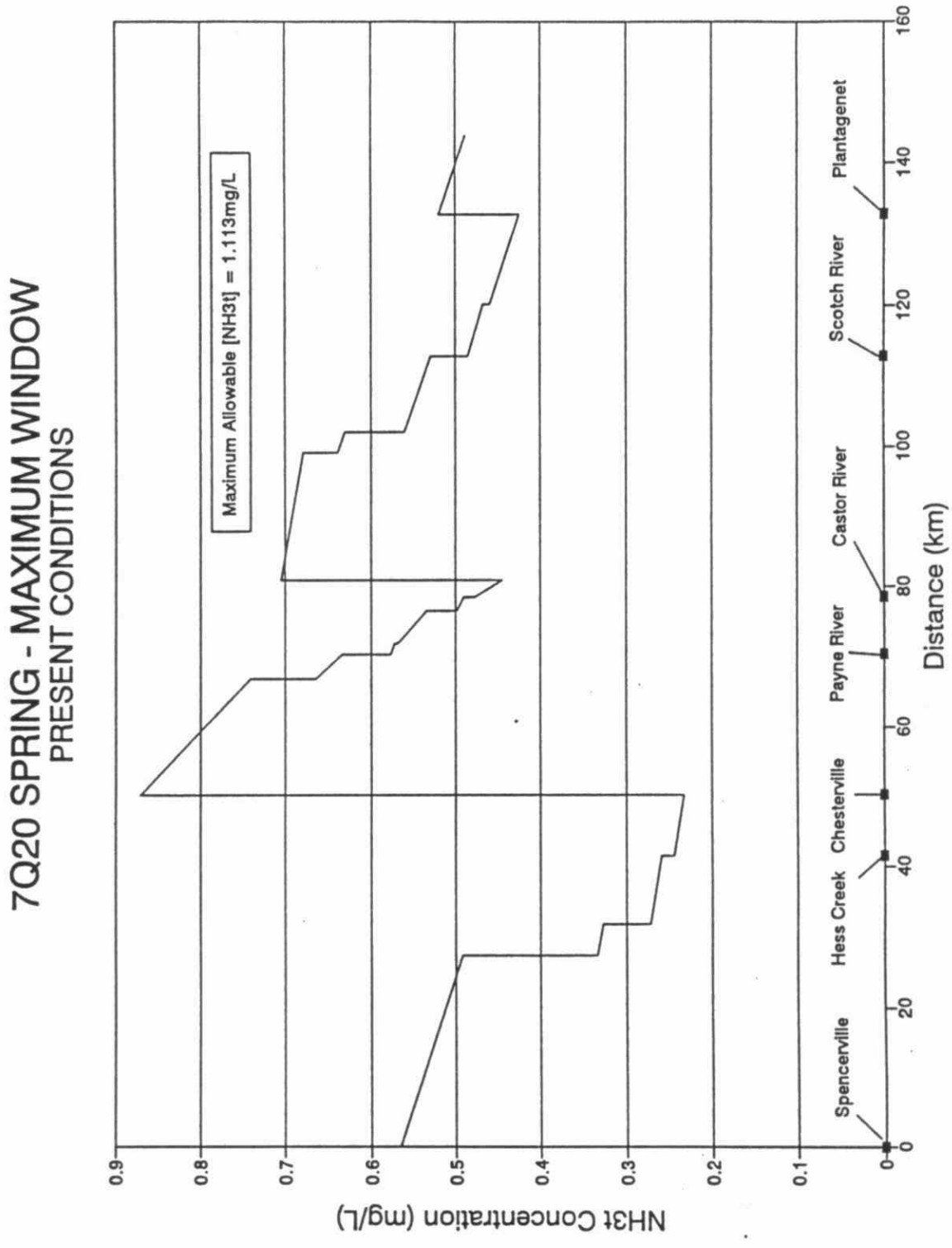

Figure 6-15 Spring 7Q20 Maximum Window NH_{3T} Profile for Present Conditions

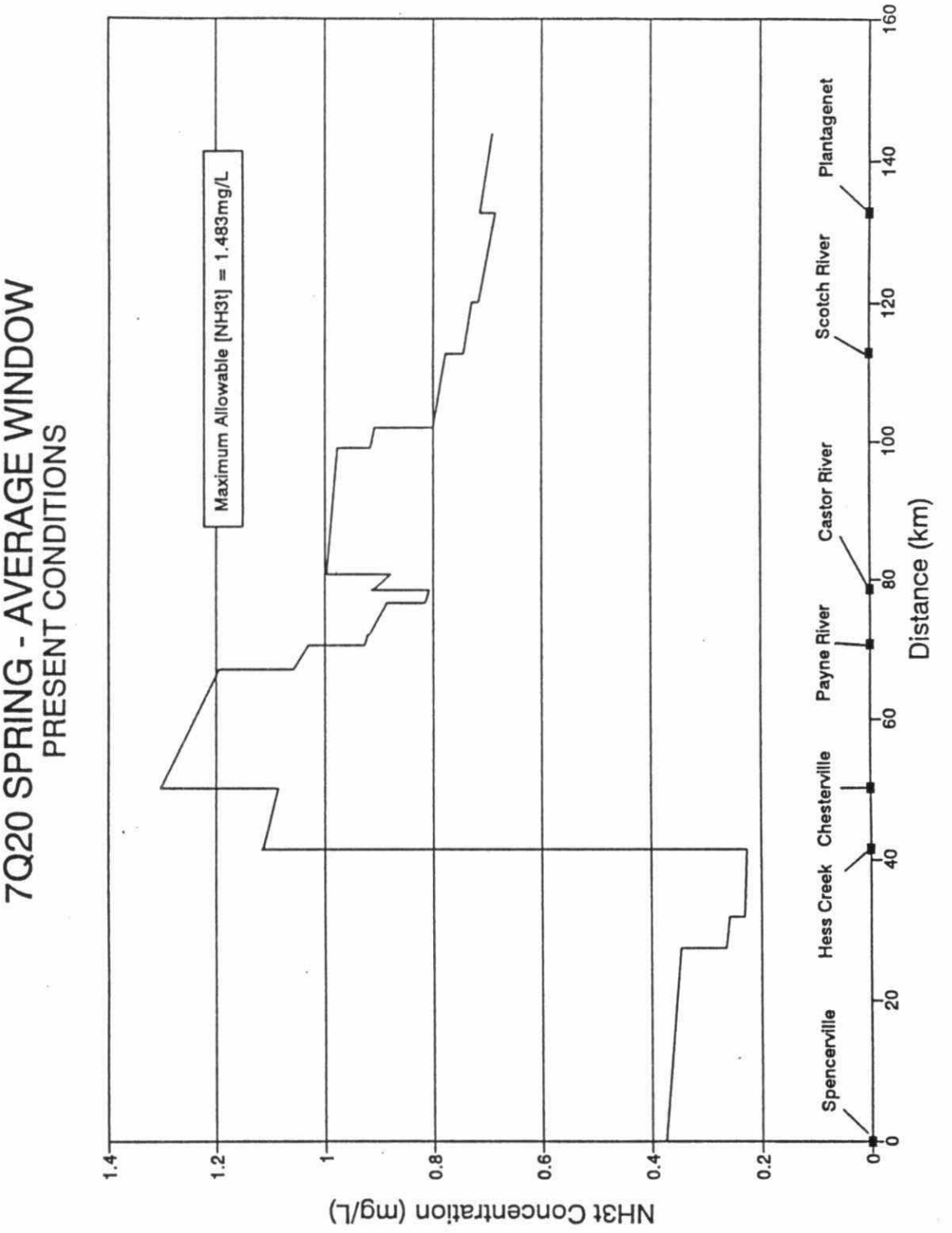

Figure 6-16 Spring 7Q20 Average Window NH_{3T} Profile for Present Conditions

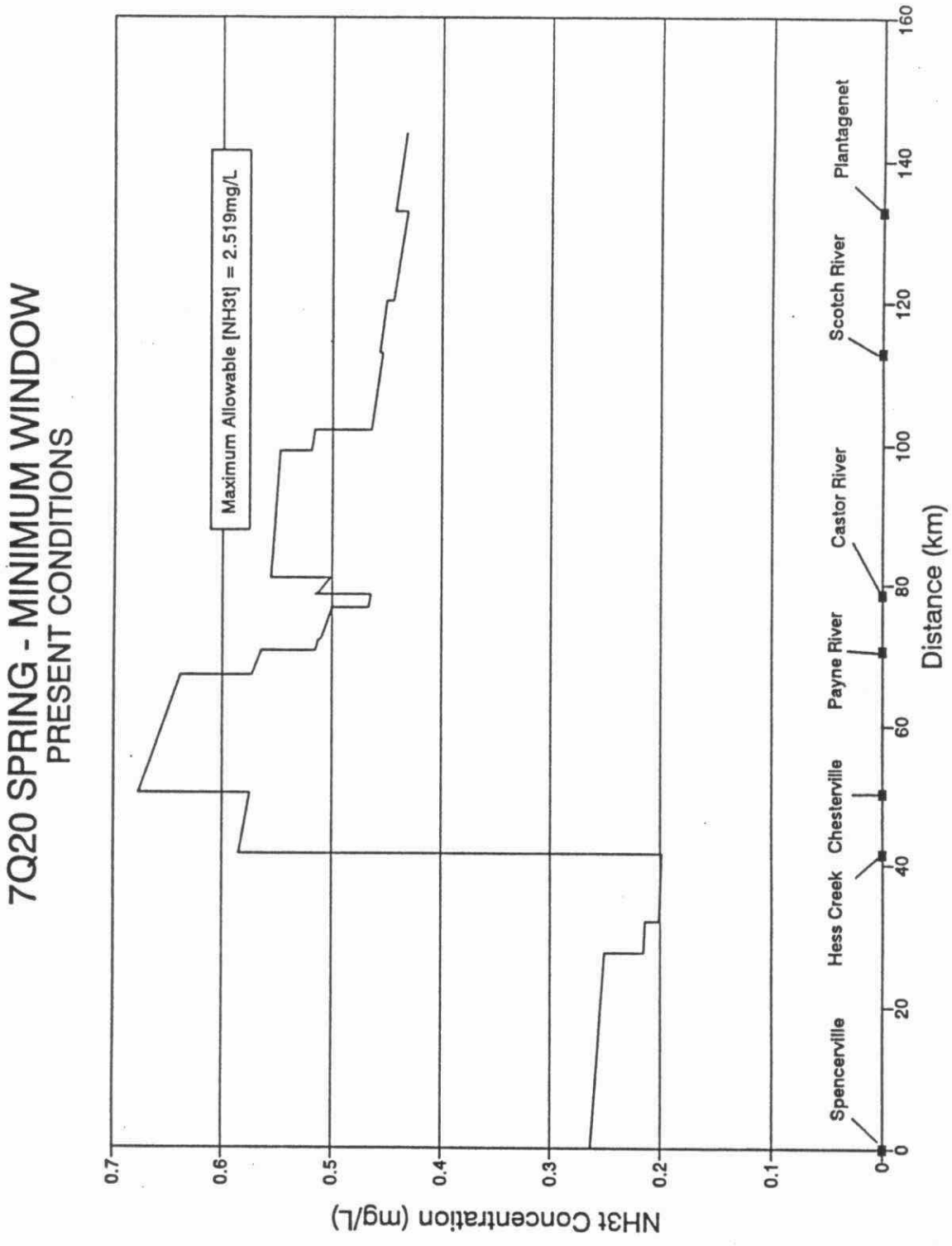

Figure 6-17 Spring 7Q20 Minimum Window NH_3 Profile for Present Conditions

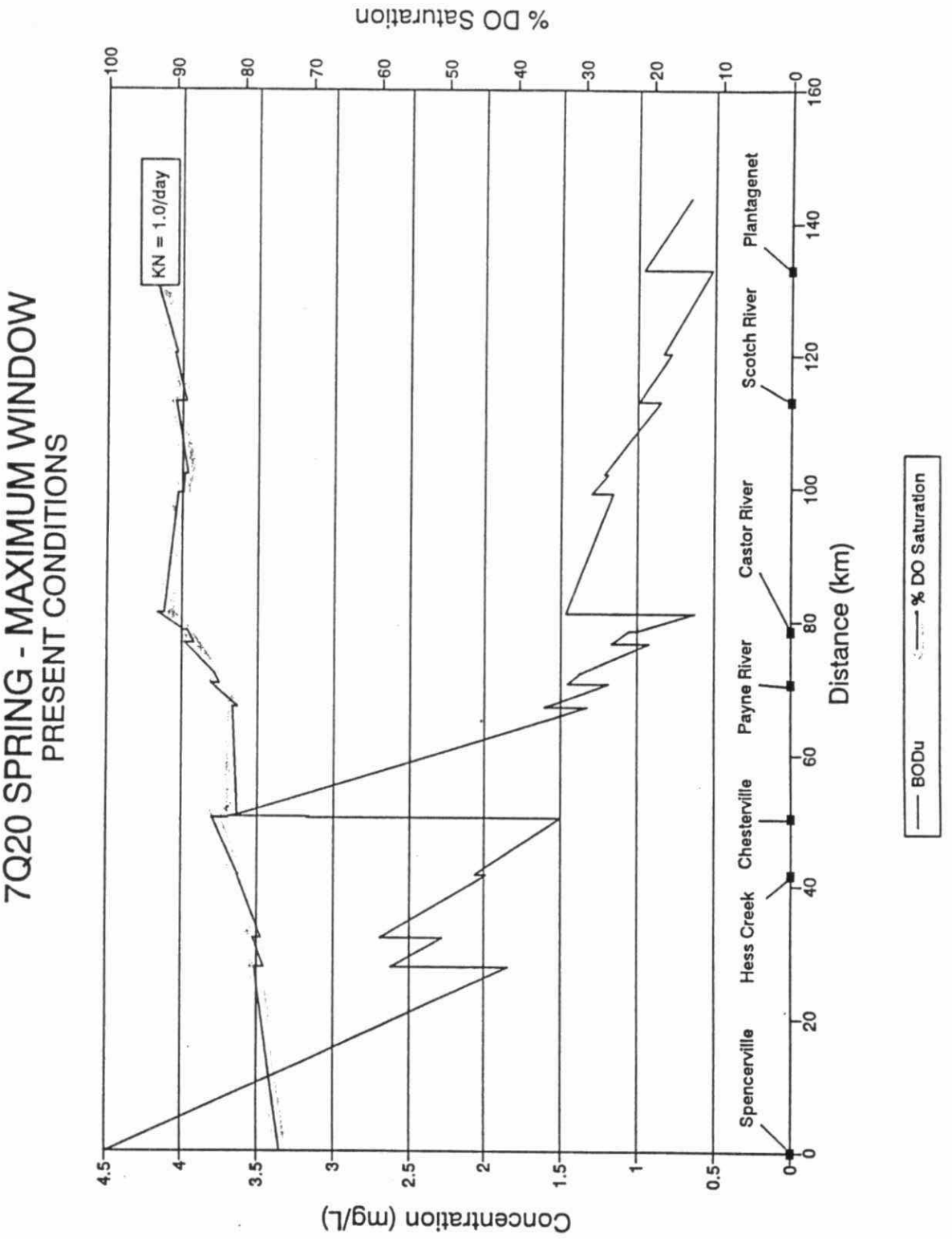

Figure 6-18 Spring 7Q20 Maximum Window DO and BOD Profiles using Coefficient Set 3

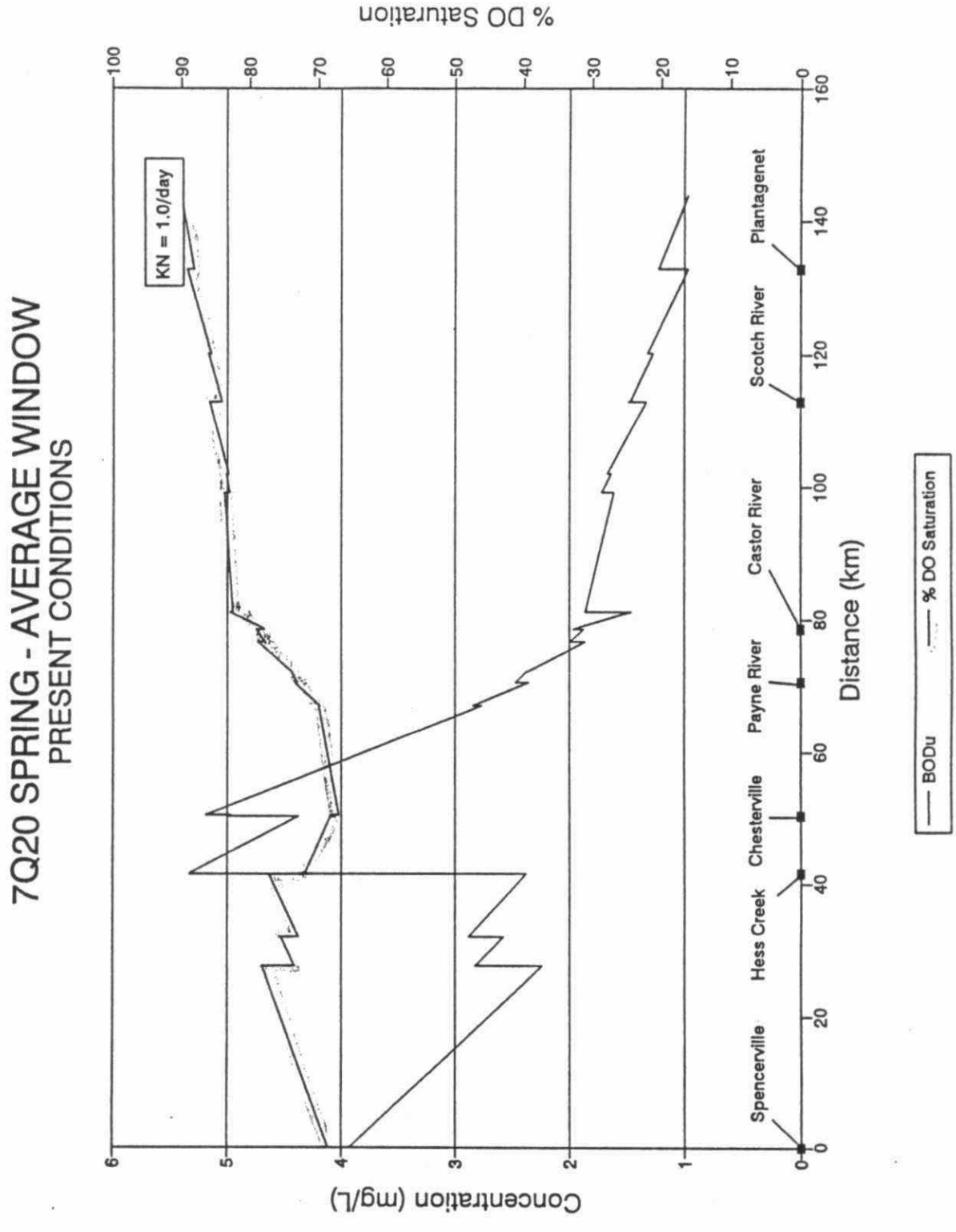

Figure 6-19 Spring 7Q20 Average Window DO and BOD Profiles using Coefficient Set 3

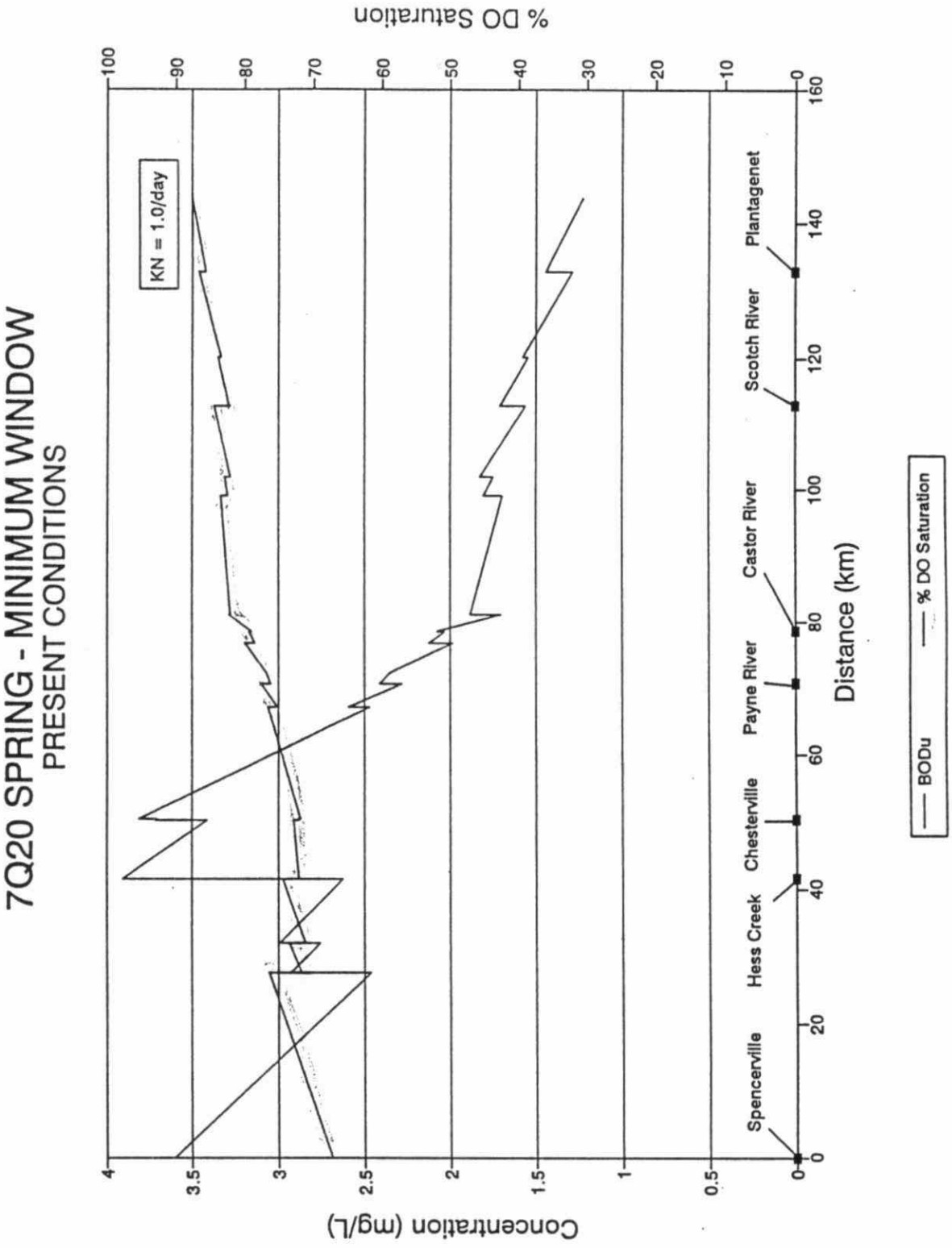

Figure 6-20 Spring 7Q20 Minimum Window DO and BOD Profiles using Coefficient Set 3

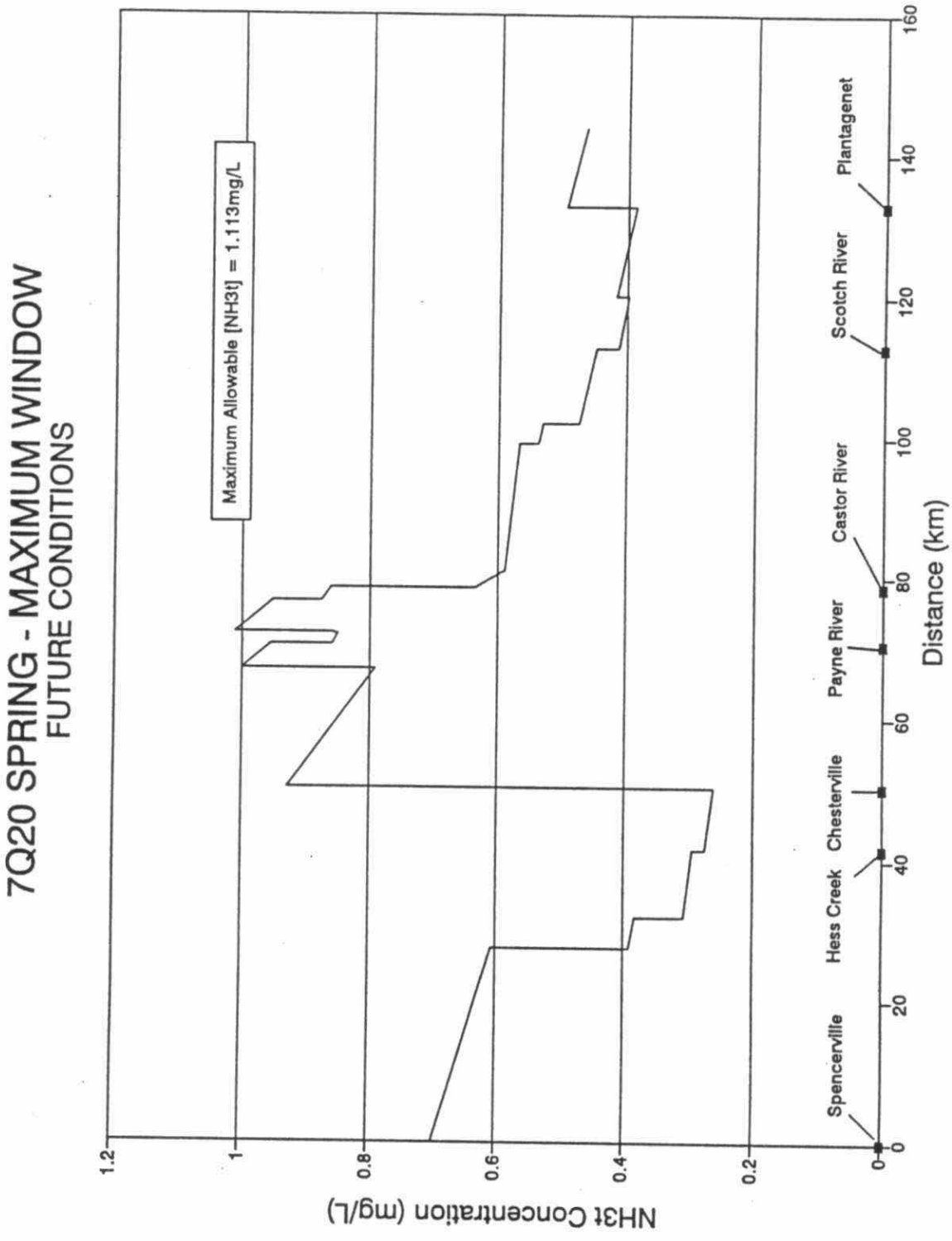

Figure 6-21 Spring 7Q20 Maximum Window NH_{3T} Profile for Future Conditions

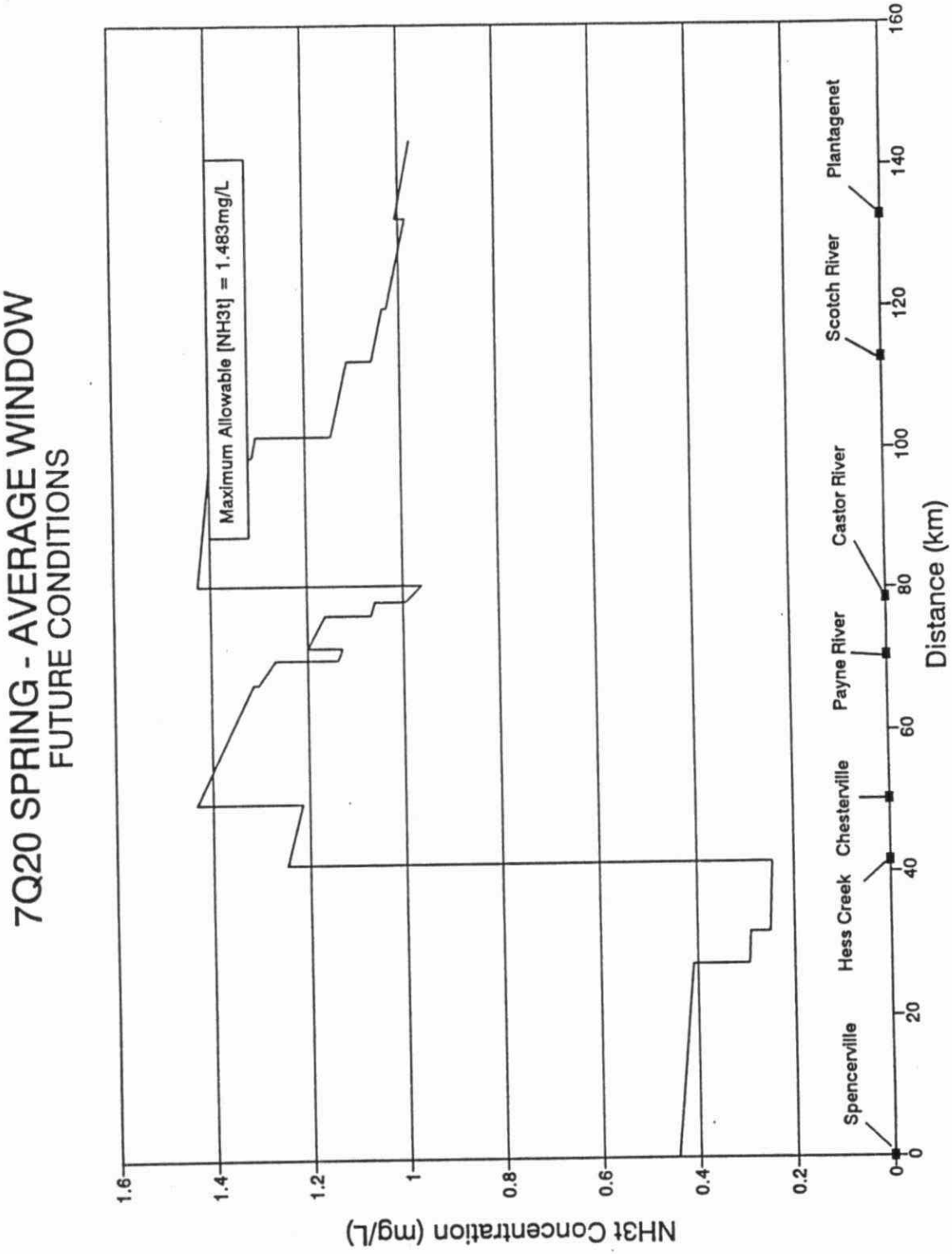

Figure 6-22 Spring 7Q20 Average Window NH_{3T} Profile for Future Conditions

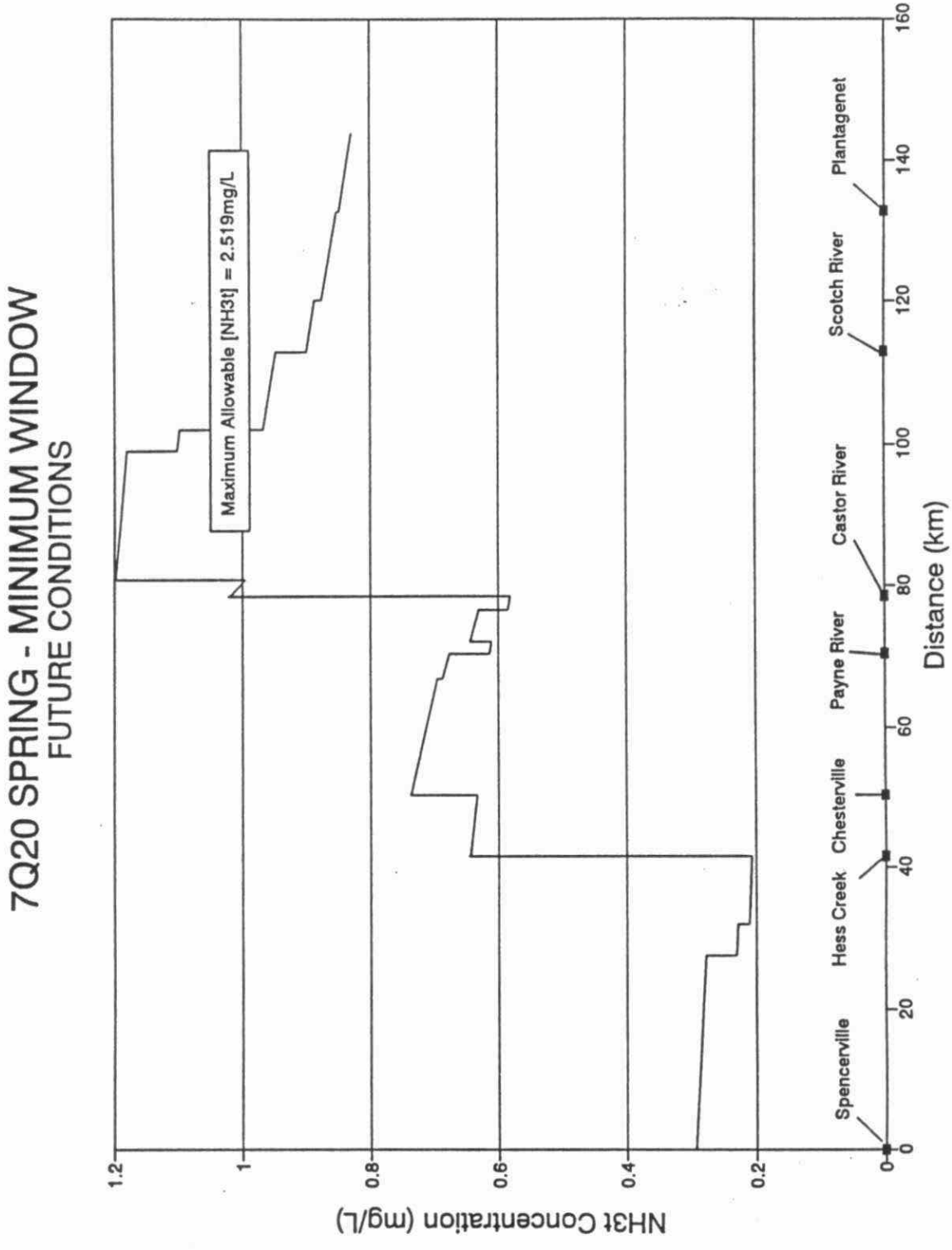

Figure 6-23 Spring 7Q20 Minimum Window NH_{3T} Profile for Future Conditions

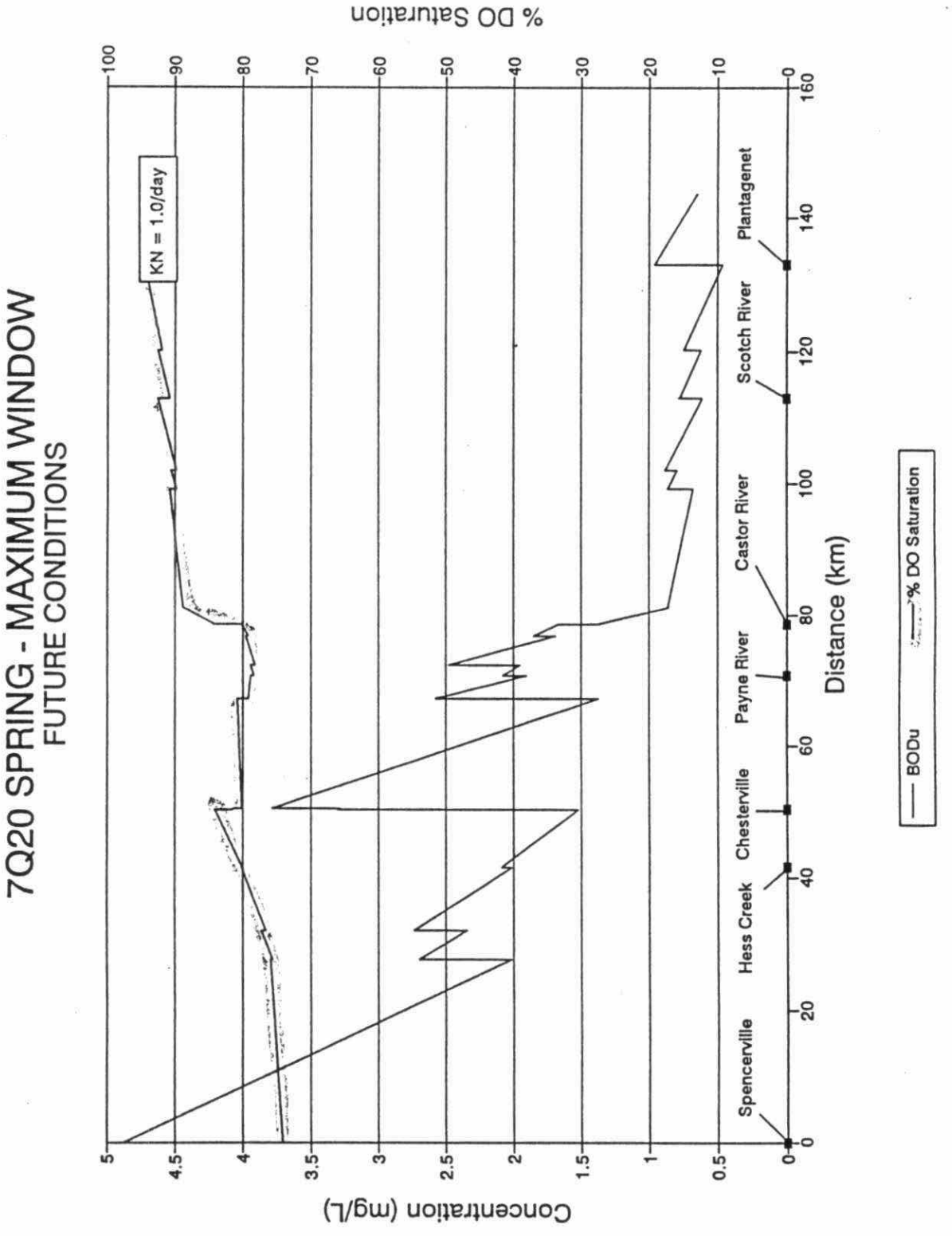

Figure 6-24 Spring 7Q20 Maximum Window DO and BOD Profiles for Future Conditions using Coefficient Set 3

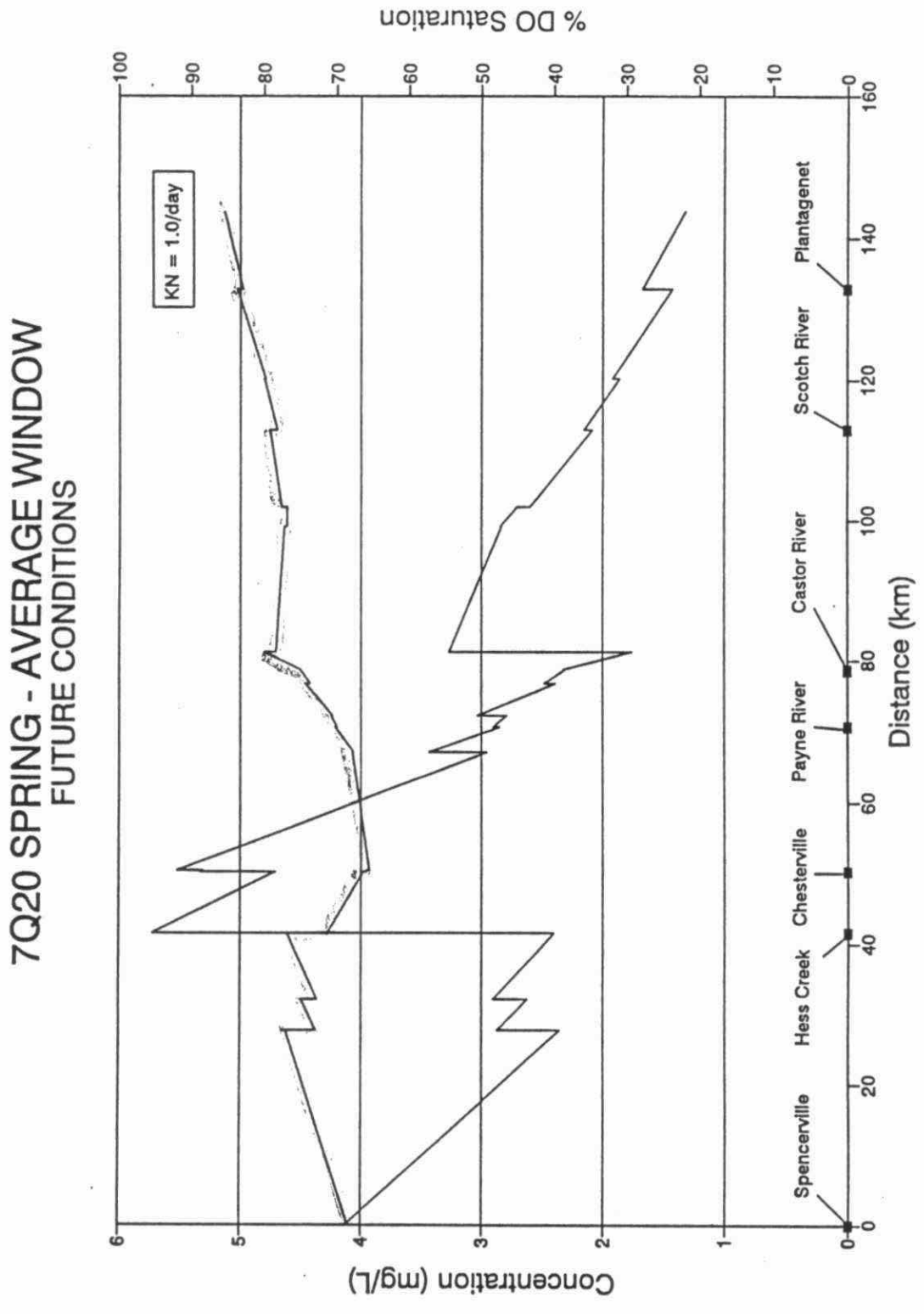

Figure 6-25 Future Condition Spring 7Q20 Average Window DO and BOD Profiles using Coefficient Set 3

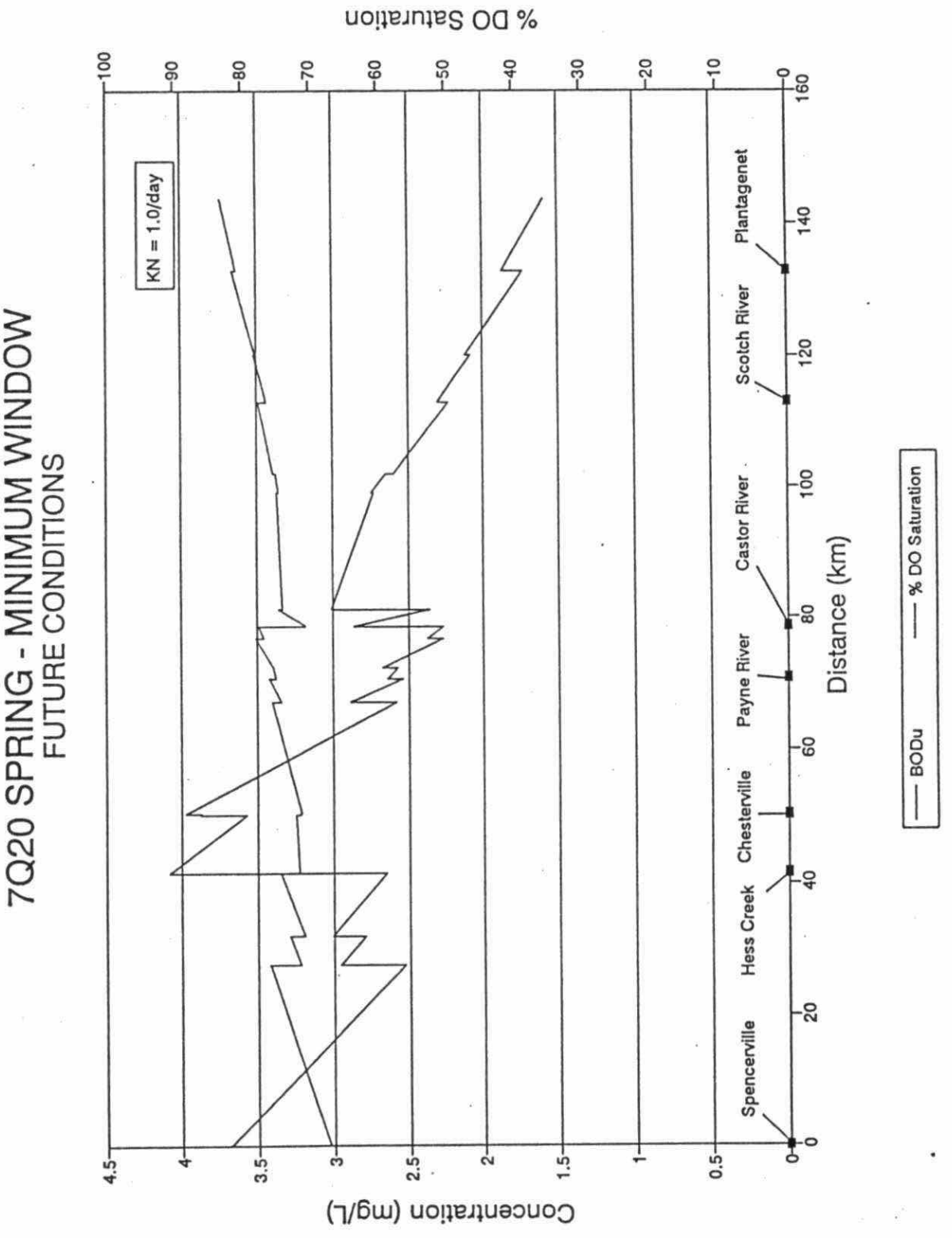

Figure 6-26 Future Condition Spring 7Q20 Minimum Window DO and BOD Profiles using Coefficient Set 3

Table 6-4 Optimum Wastewater Allocation and Treatment for Present 7Q20 Flows

Discharge Window	Point Source	Treatment[2]	Discharge Location[3]	Volume Discharged (m^3)	BOD_5 Loading (kg)	NH_{3T} Loading (kg)	Discharge Rate (m^3/s)
Max March 22 to May 13	Casselman	L	G	365000	10950	5110	0.0797
	Chesterville	L	M	296015	8880	4150	0.0646
	Nestle	I[1]	M	225862	3390	1380	0.0493
	Plantagenet	L	B	225205	6760	3160	0.0492
	Russell	L	AA	193085	5790	2710	0.0422
	Spencerville	L	Q	51100	1530	720	0.0112
Ave March 25 to April 26	Ault	I[1]	AB	193800	970	1180	0.0680
	Embrun	L	Z	474500	14200	6650	0.1664
	St-Isidore	L	S	127000	3810	1780	0.0445
	Winchester	L	N	629600	18900	8820	0.2208
Min March 30 to April 13	Maxville	L	T	59860	1800	840	0.0462
	Williamsburg	EA	AG	64100	960	320	0.0495

1. Industrial treatments are discussed in Sec. 2.7.
2. EA - extended aeration; L - lagoon (refer to Sec. 2.7 for effluent concentrations).
3. Discharge locations are nodes on Fig. 3-7.

Table 6-5 Optimum Wastewater Allocation and Treatment for Future 7Q20 Flows

Discharge Window	Point Source	Treatment[2]	Discharge Location[3]	Volume Discharged (m^3)	BOD_5 Loading (kg)	NH_{3T} Loading (kg)	Discharge Rate (m^3/s)
Max March 22 to May 13	Bourget	L	V	230200	6910	3230	0.0503
	Chesterville	L	M	318800	9560	4470	0.0696
	Crysler	L	L	219000	6570	3070	0.0478
	Fournier	L	R	50900	1530	710	0.111
	Nestle	I[1]	M	225900	3390	1380	0.493
	Plantagenet	L	B	254100	7620	3560	0.0555
	Spencerville	L	Q	69400	2080	970	0.0152
	St-Albert	L	J	138400	4150	1940	0.0302
	St-Albert Cheese	I[1]	J	22600	340	140	0.0049
Ave March 25 to April 26	Casselman	L	G	944300	28300	13200	0.3312
	Embrun	EA	Z	1551100	23300	7760	0.5440
	St-Isidore	L	S	176700	5300	2480	0.0620
	Winchester	L	N	717200	21500	10000	0.2515
Min March 30 to April 13	Ault	I[1]	AB	455000	2280	2780	0.3511
	Maxville	L	T	61300	1840	860	0.0473
	Moose Creek	EA	X	101600	1520	510	0.0784
	Russell	L	AA	865300	26000	12100	0.6677
	Williamsburg	EA	AG	70800	1060	350	0.0546

1. Industrial treatments are discussed in Sec. 2.7.
2. L - lagoon; EA - extended aeration (refer to Sec. 2.7 for effluent concentrations).
3. Discharge location nodes are indicated on Fig. 3-7.

The optimizations have been based on a determination of the minimum degree of effluent quality within the constraints of the treatment processes available and minimum distance of discharge point relocation to meet PWQOs. Chapter 8 will examine costs. It may be less costly to install a more advanced treatment process upgrade than the minimum treatment dictated by the optimization at some locations which will be beneficial to stream quality.

DO conditions generally remain at the same levels or only slightly degrade from the present to the future condition in spite of the increased number and volume of wastewater flows. This is due to a better utilization of the flood hydrograph and introduction of better treatment processes at a few locations.

6.6 DISCUSSION OF OPTIMIZATION RESULTS

The results from the optimization-modelling exercise have demonstrated that the South Nation watershed is capable of assimilating wastes generated under present or future conditions on a 30Q20 or 7Q20 basis under the constraints imposed in this study. There are only a few instances where treatment will have to be upgraded beyond lagoons or effluent discharge locations must be moved. The results and the weaknesses and strengths of the study approach are discussed below.

The locations that require more sophisticated treatment than lagoons or relocation of effluent discharge points are those on the smaller tributary streams in the system. On a 30Q20 basis there is a significant amount of assimilative capacity in general in the system because it was only necessary to spread the waste discharges through one window at present conditions and two windows at future conditions.

The lower 7Q20 flows, as expected, require a higher degree of utilization of the stream flows throughout all three windows at both present and future conditions. At future conditions, in addition to treatment upgrades and/or discharge point relocation in the drains, the Castor River is unable to accommodate waste discharges from Ault, Russell and Embrun without Embrun upgrading its treatment to extended aeration. But even under future conditions, lagoon treatment will suffice for the majority of the communities. With treatment upgrading there is still room for more growth than projected in this study in the communities and industries.

To insure that H_2S standards are met, lagoons that discharge in the spring as determined in this study, will need a post aeration cell to strip this compound.

The study has approached the problem from a necessarily conservative perspective. Ecosystem upsets are not rapidly recovered from. Deterioration of stream quality directly affects drinking water supplies and increases the need for treatment required to obtain a wholesome water supply. Recreational and aesthetic value of the streams are important. The South Nation River is tributary to the Ottawa River with consequent effects of the former on the latter.

A limited database was available for this study. A substantial amount of data are required to properly calibrate any model for water quality transformations. More

detailed stream geometry information or tracer studies are required to properly assess streamflows and travel times at particularly the low fall flows. An impressive and costly effort is required to obtain necessary data to improve estimates of rate coefficients and their variation. Where data are lacking the only reasonable approach is to make conservative assumptions.

The deterministic approach to analysis does not consider the random variation of parameters and can result in overly extreme conditions as conservative estimates of many parameters are operative at the same time. However, since discharges occur over relatively long periods of time, the probability that all variables can be at their extreme values within the discharge period increases. An environmental hiatus need only be of short duration to cause some dramatic results such as fish kill. Furthermore the occurrence of human error or system failure has not been factored into the analysis. A stochastic analysis of parameter variation would be a valuable complementary study to the present effort.

In this study it was assumed that all treatment processes would be discharging effluent at the lower limits (higher concentrations) of effluent quality. Stream background concentrations were taken at 75 percentile values which means that 25 percent of the time, conditions are worse than assumed for the modelling. The limiting values of the rate coefficients were those at the extreme end of ranges judged to be reasonable from other studies. The one in 20 year flow has been accepted for a long time as a reasonable basis for design of wastewater systems to protect water quality on a sustained basis. A stochastic analysis and more data on the system would aid in determining the deterministic modelling constraints (such as which percentile values to use for stream background concentrations and rate coefficients) that would result in a 5% rate or any rate of failure. (A 5% rate of failure is implied by using 1 in 20 year stream flows.)

Some modelling constraints are limiting while others tend to be liberal. A stochastic analysis will improve the understanding of the overall interaction of the variables and provide a measure of the overall variability in water quality.

It is clear from examination of Figs. 3-2 and 3-4 that the constraint of constant discharge with the streamflow at or above the minimum flow in a window does not account for a large volume of water passing through the streams. For most of the windows the actual volume of in the stream is more than double the volume of water assumed to be available in this study. Variable rate discharge can be used to further take advantage of the actual volume of water available in the streams. The concept of real time control was discussed in Sec. 5.2 and dismissed because it requires a more sophisticated control and monitoring system. Evaluation of these requirements was beyond the scope of this study. For most of the point sources, this study has demonstrated that real time control is not necessary. However, for communities that must upgrade or relocate effluent discharge points, real time control may provide the opportunity for the communities to retain lagoon treatment and/or discharge their wastes into the nearest stream.

There will be costs associated with the implementation of real time control but they may be considerably less than the costs of advanced treatment and effluent relocation dictated by the current study. There will also be a concomitant reduction in the safety factor if real time control is implemented and the system is pushed closer to its limits.

Another limitation of this study was the confinement of the study to the spring and fall seasons. It is clear that more sophisticated treatment is required at some locations under the constraints of this study but the implementation of advanced treatment may not need to be as extensive as found here if a careful evaluation of streamflow variation throughout the year is conducted.

The background concentrations of ammonia from NPS impress a severe limitation on the assimilative capacity of the streams for point source discharges. Background concentrations of ammonia consume more than half of the allowable stream concentration in the month of April. Certainly some reduction of background concentrations of ammonia will be gained by installing sewerage and wastewater treatment for unsewered communities although the reduction may be very small or even negligible.

There is probably a much higher potential for reduction in the background concentrations by considering improvements in agricultural practices. Excessive fertilizer application rates, livestock direct access to streams, lack of buffer strips and other detrimental practices have been observed in the watershed. A study of the occurrence of these practices will be able to provide some estimate of reductions that may be expected.

As shown in Chapter 4, river quality has improved during the past 12 years although phosphorus is an exception to this improvement. The installation of sewage treatment works has probably contributed to the improvement. Changes in agricultural practice may also have been a contributing factor but this seems to be less likely since phosphorus background concentrations have not changed. This implies that fertilizer application and other NPSs for phosphorus have not changed. The issue of background concentration change at low flows was unable to be resolved with available data. It is logical to assume that less runoff and concomitant streamflow reduction would elevate background concentrations of pollutants if agricultural practices do not change during a dry year. Using the 75 percentile stream concentrations as a basis for estimating the background concentrations would provide some allowance for this situation but it is not clear to what extent the issue is addressed.

This study has allocated wastewater based on the minimum amount of wastewater treatment and the least amount of relocation for the point sources for the watershed as a whole. The overriding principle was to allocate the wastewater at the least cost to the watershed, allowing each community or industry to expand to their projected future conditions. This paradigm does not take into account disparities in growth rate among the communities and does not begin to address secondary issues such as the socioeconomic ramifications of the policy. Under the projected growth in populations and industries utilized in this study there is relatively only slight demands put on the

watershed for treatment upgrades from lagoons. But at ultimate growth conditions where stream assimilative capacity is utilized to the maximum extent, these issues will become very important.

There are a multitude of ultimate population growth scenarios that will be feasible. Population growth could be allowed based on the same geometric growth rate, the same arithmetic growth rate, or a policy that more fully utilizes the stream assimilative capacity. The solutions derived here are limited by the stream reaching critical PWQO values at various locations but there are other schemes that would keep the streams at or nearer their limits over great reaches- at a cost. Socioeconomic ramifications include trade-offs between industrial or agricultural growth and population expansion and population demographics in the region. Overall environmental harmony, aesthetic considerations and legal issues are other considerations. This study has only addressed wastewater allocation, water allocation is another issue.

The above are only a few of the issues that need to be considered in developing wastewater allocation guidelines. There is a need to develop a comprehensive water and wastewater allocation policy for the province. This study is progressive in that it examines the watershed as an integrated unit but it is still short of a holistic evaluation of water and related issues.

Concerns regarding the 7Q20 and 30Q20 streamflow bases for conducting this study are raised in Sec. 5.2. Studies should be conducted to affirm one flow condition or the other as the most reasonable basis to protect the receiving waters and in particular, the aquatic life within them. The lower flows under 7Q20 conditions produce significant increases in measures required to maintain PWQO in particularly the small streams.

A qualifying remark must be noted for the remote communities. The dilutions available at the upstream reaches are small and the effluent from the treatment facility dominates the mixed flow. But the analysis of extreme conditions was based on measurements particularly of pH, for the streams. When effluent from the treatment facility dominates the mixed flow, the pH and buffering capacity of the effluent will more highly influence the pH of the mixed water. For the case of lagoons an effluent pH of 8.5 (Table 2-9) is specified. Using this pH instead of the 75th percentile stream values will not result in any favourable change for the municipalities.

However, for Ault, the treatment process employed there does reduce the effluent pH to remove phosphorus and the final pH is well below 8. This will considerably change the dilution requirement to meet the ammonia PWQO. However, there are still ramifications for the other downstream communities of Russell and Embrun. Also after streamflow is augmented by confluence and groundwater recharge to the point where the pH reaches typical stream values, the ammonia contribution from Ault will then cause an un-ionized ammonia violation although the violation will occur at some distance downstream from the Annable Drain into which Ault would discharge.

SECTION 7

MIXING ZONE ANALYSIS AND PHOSPHORUS CONSTRAINTS

MIXING ZONE ANALYSIS AND PHOSPHORUS CONSTRAINTS

The analysis described in Chapter 6 assumed that the effluent was completely mixed with the receiving water at the point of discharge. In fact, the effluent and stream may take a significant amount of time to attain the fully mixed condition. An analysis of mixing zones was required for this study. Existing Certificate of Approvals, C of A's, do not address mixing zones, however, future specifications may adopt mixing zone limitations.

A comprehensive mixing zone analysis would be required to accurately define the waste plume and minimum zones of passage required to achieve the regulatory concentration limits. However, the minimum zones of passage may be approximated through application of relatively simple mixing analysis. This approximate approach, which was adopted in this investigation, provides an adequate definition of mixing zones.

A typical plume is illustrated in Fig. 7-1 with a few explanatory notes. As the plume moves downstream, mixing of the waste effluent with the receiving water contributes to lateral and vertical spreading of the plume. For a discharge located in the middle of the stream the form of the plume will generally be Gaussian, with the peak concentration occurring along the plume centre-line. As the plume spreads the centre-line concentration is reduced and the extent of the cross-sectional area impacted is increased. Eventually, the limits of the plume impinge on the channel boundaries and a fully mixed condition will be achieved at some point further downstream. For a shoreline discharge the mixing of the waste discharge and the river is similar, however, the plume spreads towards the opposite shore.

For any particular contaminate plume, contours of equivalent constituent concentrations can be determined. A contour which corresponds to a PWQO objective will encircle a region in which the PWQO objective is exceeded. An example PWQO contour of this form is illustrated on Fig. 7-1. The region outside of the enclosed area represents the zone of passage. The location at which the zone of passage is a minimum represents the minimum zone of passage and the MOE requires that the minimum zone of passage be at least 75% of the total available flow.

If dilution capacity under fully mixed conditions is not adequate to meet the PWQO, then a mixing zone analysis is not warranted.

7.1 CORMIX1 BASED CRITERION

Several applications of the mixing zone analysis model CORMIX1 have led to the conclusion that under certain conditions, if acceptable dilution can be achieved with 25% of the total flow, then it is likely that the concentration limit will be achieved with a 75% zone of passage. This criterion was applied to all discharges for the present and future solutions of both the 7Q20 and 30Q20 flows to establish whether or not the discharge complied with the MOE 75% minimum zone of passage rule.

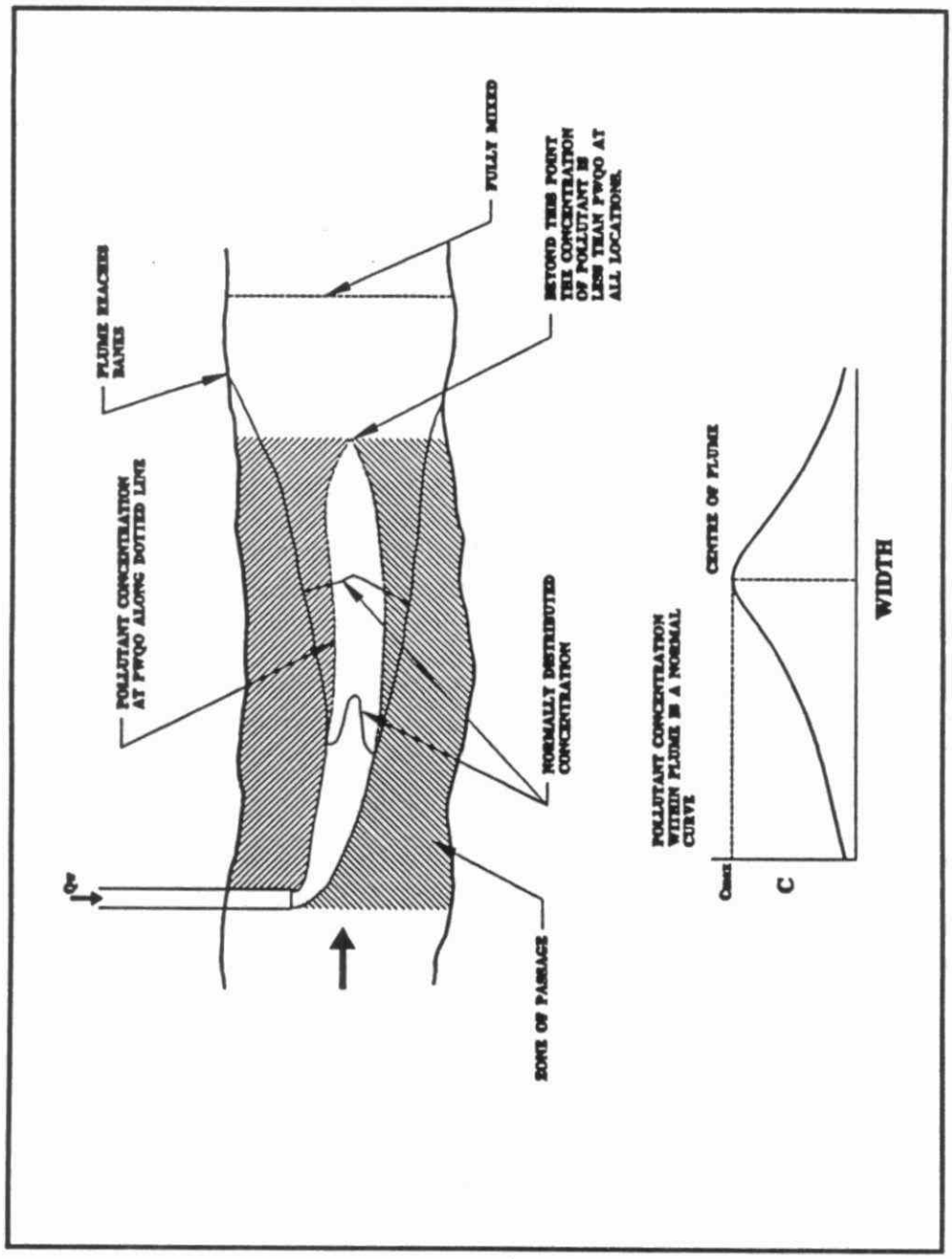
FULLY MIXED
BEYOND THIS POINT THE CONCENTRATION OF POLLUTANT IS LESS THAN PWQO AT ALL LOCATIONS.
PLUME REACHES BANKS
POLLUTANT CONCENTRATION AT PWQO ALONG DOTTED LINE
NORMALLY DISTRIBUTED CONCENTRATION
ZONE OF PASSAGE
CENTRE OF PLUME
POLLUTANT CONCENTRATION WITHIN PLUME IS A NORMAL CURVE
C
WIDTH

Figure 7-1 Typical Waste Plume

The dilution formula defining the completely mixed concentration, C_m, can be expressed as:

$$C_m = \frac{C_r Q_r + C_w Q_w}{Q_r + Q_w} \quad (7\text{-}1)$$

where

Q_r = river flow
Q_w = wastewater flow
C_w = discharge concentration
C_r = upstream river concentration

Equation 7-1 may be adapted to yield an estimate of the concentration corresponding to a 75% zone of passage, C_1. Based on the criterion defined above, if this concentration is less than or equal to the MOE objective concentration, the 75% minimum zone of passage rule will have been met. Equation 7-2 illustrates the revised dilution formula which provides an estimate of C_1.

$$C_1 = \frac{C_r[(1-ZP)(Q_w+Q_r) - Q_w] + C_w Q_w}{(Q_r+Q_w)(1-ZP)} \quad (7\text{-}2)$$

where

ZP = minimum zone of passage, in this case 75%
C_1 = limiting concentration (PWQO)

All spreadsheet models have been retrofitted to automatically calculate wether or not a discharge complies with the 75% zone of passage objective with respect to NH_{3T}. Since the concentration of NH_{3T} just upstream of a node (C_r), the river flow at the node (Q_r), the wastewater flow (Q_w), and the discharge concentration of NH_{3T}, (C_d), are all known, equation 7-2 may be applied to estimate ZP. The solution spreadsheets automatically calculate a reasonable approximation of the zone of passage, ZP, based on ammonia toxicity and equation 7-2.

Printouts of the spreadsheet solutions are provided in Appendix G and Tables 7-1 and 7-2 provide a summary of the results for the 30Q20 and 7Q30 flow conditions respectively. The discharges which meet the 75% zone of passage for ammonia are indicated with a "P" while an "F" indicates a discharge which failed to meet the 75% zone of passage rule.

Table 7-1 75% Zone of Passage Compliance for 30Q20 Present and Future Conditions

DISCHARGER	NODE	PRESENT			FUTURE		
		WINDOWS			WINDOWS		
		MAX	AVG	MIN	MAX	AVG	MIN
AULT	AB	F[1]	P[2]	P	F	F	F
BOURGET	V	NOL[3]	NOL	NOL	P	P	P
CASSELMAN	G	P	P	P	P	P	P
CHESTERVILLE	M	P	P	P	P	P	P
CRYSLER	L	NOL	NOL	NOL	P	P	P
EMBRUN	Z	F	P	P	ND[4]	F	F
FOURNIER	R	NOL	NOL	NOL	F	P	P
MAXVILLE	T	F	F	P	F	F	P
MOOSE CREEK	X	NOL	NOL	NOL	ND	F	F
NESTLE	M	P	P	P	P	P	P
PLANTAGENET	B	P	P	P	P	P	P
RUSSEL	AA	P	P	P	F	F	F
SPENCERVILLE	Q	P	P	P	P	P	P
ST ALBERT	J	NOL	NOL	NOL	P	P	P
ST ALBERT CH.	J	NOL	NOL	NOL	P	P	P
ST ISIDORE	S	F	P	P	F	P	P
WILLIAMSBURG	AG	F	F	P	F	F	P
WINCHESTER	N	P	P	P	F	P	P

1. F = Fail
2. P = Pass
3. NOL = Not on line
4. ND = On Line, but not discharging throughout the duration of the window

Table 7-2 75% Zone of Passage Compliance for 7Q20 Present and Future Conditions

		PRESENT			FUTURE		
		WINDOWS			WINDOWS		
DISCHARGER	NODE	MAX	AVG	MIN	MAX	AVG	MIN
AULT	AB	ND[1]	F[2]	F	ND	ND	F
BOURGET	V	NOL[3]	NOL	NOL	F	F	P[4]
CASSELMAN	G	F	P	P	ND	F	P
CHESTERVILLE	M	F	F	P	F	F	P
CRYSLER	L	NOL	NOL	NOL	F	F	P
EMBRUN	Z	ND	F	P	ND	F	F
FOURNIER	R	NOL	NOL	NOL	P	P	P
MAXVILLE	T	ND	ND	F	ND	ND	F
MOOSE CREEK	X	NOL	NOL	NOL	ND	N.D.	F
NESTLE	M	F	F	P	F	F	P
PLANTAGENET	B	P	P	P	P	P	P
RUSSEL	AA	F	F	P	ND	ND	F
SPENCERVILLE	Q	F	P	P	F	P	P
ST ALBERT	J	NOL	NOL	NOL	P	P	P
ST ALBERT CH.	J	NOL	NOL	NOL	F	P	P
ST ISIDORE	S	ND	F	P	ND	F	F
WILLIAMSBURG	AG	ND	ND	F	ND	ND	F
WINCHESTER	N	ND	F	P	ND	F	P

1. ND = On line but not discharging throughout the duration of the window
2. F = Fail
3. NOL = Not on line
4. P = Pass

Tables 7-1 and 7-2 indicate that for 30Q20 flow conditions, most nodes achieve a 75% zone of passage or greater for the present conditions, and those that do not are located on tributary streams. For the 30Q20 future conditions, a greater number of tributary stream dischargers do not achieve the 75% zone of passage, and one discharge (Winchester) on the South Nation River fails for the maximum window.

For present 7Q20 flow conditions, 5 out of 6 discharges fail during the maximum window, 7 out of 10 fail in the average window, and 3 out of 12 fail in the minimum window. The future condition is worse: 6 out of 9 discharges fail during the maximum window, 8 out of 13 fail in the average window and 7 out of 18 fail in the minimum window. A number of the failures occur in the South Nation River, and the majority of discharges released in tributary streams failed to achieve the 75% zone of passage.

Implicit in this analysis is the assumption that upstream of a node completely mixed conditions prevail. Therefore, the effect of overlapping plumes was not addressed. It should be emphasized that this analysis is not based on a comprehensive mixing zone analyses (CORMIX1). The results do however provide a reasonable approximation of the zone of passage, but should be interpreted with some degree of caution and judgement.

7.2 PHOSPHORUS

Elevated levels of phosphorus are often associated with eutrophication. Phosphorus, an essential nutrient for all life forms, is generally the limiting nutrient for growth in aquatic systems. Phosphorus is a highly immobile element in the environment due to the low solubility products associated with many phosphorus compounds. Phosphorus combines with many relatively abundant metals forming relatively insoluble compounds which readily precipitate out of solution.

Since primary productivity is usually correlated with available phosphorus, anthropogenic sources have a significant impact on aquatic systems.

7.2.1 Current Phosphorus Loads

Phosphorus loadings at future flow conditions are not expected to increase, and a reduction in current loadings is desirable. The regional MOE policy currently requires that the total phosphorus concentration, TP, of treated wastewater, be less than or equal to 1 mg/L. Future growth must maintain TP loads at or below present C of A loads.

Average phosphorus loadings based on recent available data for communities with existing treatment facilities are given in Table 7-3. Facilities which are not listed either had no available data or have not yet discharged effluent.

Acceptable phosphorus loads, as defined by C of A specifications, are given in Table 7-4. For facilities with no C of A specified phosphorus effluent concentration, a value of 1.0 mg/L TP was assumed. Based on the TP limits listed in Table 7-4, the communities of Plantagenet and Winchester and the industry Ault Foods are at the limit of loads defined by the C of A.

As discussed in Chapter 2, phosphorus reduction (usually with alum addition) has only been initiated recently. Furthermore, no TP effluent requirements were in place for

many of the facilities, and in some cases no special efforts were taken to minimize effluent phosphorus concentrations. Therefore, average historical data for phosphorus may yield loadings which are greater than present.

Comparing Tables 7-3 and 7-4 reveals that for the communities with a TP specification (Plantagenet and Russell; Maxville has yet to discharge), Plantagenet has exceeded the load specified by the C of A. This is not the case for the two industries which are both well over the C of A specified phosphorus loads. Efforts are under way to improve this situation.

For the assumed C of A TP limit of 1 mg/L, only the communities of Casselman and Embrun would have historically met the loading requirement. Chesterville and Winchester are well over the assumed C of A limit, while St. Isidore is approximately at the limit.

Table 7-3 Average Phosphorus Loads from Point Sources

Location	No. of Years of Record	Average Loading[1] kg/yr
Casselman	5	166
Chesterville	5	483
Embrun	1	470
Plantagenet	5	394
Russell	5	138
St. Isidore	5	247
Winchester	5	1416
Ault	2	391
Nestles	4	1412

1. Loads were obtained from Appendix E.

7.2.2 Future Phosphorus Loads

The allowable TP concentrations in effluent at future conditions based on historical phosphorus loads and C of A specifications (assumed or otherwise) are given in Table 7-5. The estimated annual waste volumes necessary for Table 7-5 were obtained from Sec. 2.6.2. Historical TP concentrations range from 0.16 - 1.0 mg/L, while C of A TP concentrations range from 0.36 - 1.0 mg/L. The C of A permits an increase in flow to a point where the loads indicated in Table 7-4 are achieved, however, TP concentrations cannot exceed 1 mg/L.

Table 7-4 Phosphorus Loads Based on Certificates of Approval

Location	Actual Annual Volume m^3/yr	C of A Annual Volume m^3/yr	C of A Criteria mg/L	Actual Annual Load kg/yr	C of A Annual Load kg/yr
Casselman	365000	491290	(1.0)[1]	(365)	(491)
Chesterville	296015	379600	(1.0)	(296)	(380)
Embrun	474500	656270	(1.0)	(475)	(656)
Maxville	59860	164250	1.0	60	164
Plantagenet	225205	215350	1.0	225	215
Russell	193085	365000	1.0	193	365
Spencerville	51100	116000	(1.0)	(51)	(116)
St. Isidore	127020	239075	(1.0)	(127)	(239)
Williamsburg	64094	445300	(1.0)	(64)	(445)
Winchester[4]	629625	629625	1.0	630	648 (SA) 469 (A)
Ault[2]	193830	161700	1.0	194	167
Nestle[3]	225857	746425	1.0	226	746

1. Parentheses denote that no P specification was included in the C of A. A value of 1 mg/L TP (current MOE policy) was assigned.
2. Ault's flows were based on the years 1988 and 1989.
3. Nestle's flows were based on the years 1986-1989.
4. Winchester is allowed a load of 648 kg/yr for semi-annual discharge and a load of 469 kg/yr for an annual discharge.

Because of inconsistent phosphorus removal practices, for past and present, there is a wide range of future TP effluent concentrations that would be generated based on either historical or C of A loads. Table 7-5 does not include proposed treatment facilities (Bourget, Fournier, and others).

Since the majority of wastewater in communities where treatment facilities are proposed are disposed of through septic tank systems it is unlikely that the corresponding phosphorus contributions to the streams are significant. Converting these communities to sewerage will undoubtedly increase phosphorus contributions to receiving waters, however, the impact on the river will be small since point sources only account for a maximum of 7% of the total river phosphorus load (Sec. 4.4).

Table 7-5 Allowable Total Phosphorus Concentrations for Future Conditions

Location	1991		2011		
	Actual Load kg/yr	C of A Load[1] kg/yr	Annual Volume[3] m^3	P Based on Historical Load mg/L	P Based on C of A Load mg/L
Casselman	166	(491)[2]	944300	0.40	(0.52)[6]
Chesterville	484	(380)	318791	1.0[5]	(1.0)[6]
Embrun	471	(656)	1551104	0.30	(0.42)
Maxville	NA	164	61320	—	1.0[6]
Plantagenet	394	215	254138	1.0[5]	0.85
Russell	139	365	865269	0.16	0.42
Spencerville	NA[4]	116	91952	—	1.0[6]
St. Isidore	248	(239)	176684	1.0[5]	(1.0)[6]
Williamsburg	NA	(445)	70755	—	(1.0)[6]
Winchester	1416	648[7]	717152	1.0[5]	0.90
Ault	391	167	455000	0.86	0.36
Nestles	1412	746	226000	1.0[5]	1.0[6]

1. C of A loads were taken from the last column of Table 7-4.
2. Parentheses denote that the present load was based on the C of A hydraulic capacity and an assumed effluent TP of 1 mg/L.
3. Annual volumes are taken from Table 2-8.
4. NA - not available.
5. Historical loads would allow concentrations greater than 1 mg/L but MOE policy will not allow concentrations exceeding this limit.
6. Current C of A annual loads would allow concentrations at greater than 1 mg/L to maintain the current load but MOE policy would not allow TP concentrations greater than 1 mg/L.
7. The maximum load specified in the current C of A was used.

With present background concentrations of phosphorus in the South Nation River system there is no assimilative capacity for phosphorus (Chapter 4). New point sources will have to achieve low effluent TP concentrations along with concurrent reductions in NPS to return the rivers to a desirable state with respect to this pollutant.

Achieving effluent TP concentrations less than 0.4 mg/L is beyond the realm of reasonable expectation with any of the proposed treatment processes. The New

Hamburg process is the most viable process to maintain TP values at this limit. Alum addition and controlled effluent discharge with a conventional lagoon can achieve effluent TP concentrations approaching 0.4 mg/L. However, a realistic value for lagoons with continuous alum addition is 1.0 mg/L as indicated in Table 2-9. Any lagoon can be subject to an algal bloom as a result of the high nutrient concentrations. Discharge of effluent with high algae content will increase effluent TP concentrations.

Effluent from the New Hamburg process is filtered, preventing the discharge of effluent with high algae content. Alternatively, an algicide (some of which contain copper compounds) can be used to limit or destroy algae blooms. Unfortunately, the use of algicides may pose other threats to stream quality and are not generally recommended.

Based on historical loadings, Casselman, Embrun, and Russell would be required to install the New Hamburg process if their populations are to expand to levels projected in Chapter 2. In fact, Russell would be required to implement additional measures to remove phosphorus or otherwise restrict its population growth.

From existing C of A specifications, Embrun and Russell would be required to install the New Hamburg process to maintain a loading in compliance with current approvals at their projected populations.

SECTION 8

COST ANALYSIS

COST ANALYSIS

To evaluate the economic feasibility of the proposed facility upgrade options, capital and operating costs were estimated. All costs presented in this report are useful for a comparative evaluation of treatment options only and are not to be used for budget planning. The costs were estimated on a uniform basis for each community. Site surveys, wastewater quality analyses and other data collection required to establish firm capital and operating costs were outside the scope of this study. Any design criteria required to estimate costs are based on MOE guidelines (MOE, 1984).

At present all communities with wastewater treatment systems rely on conventional facultative lagoon facilities. The adequacy of the existing treatment facilities to serve the existing population on an hydraulic basis is summarized in Table 8-1. Only Plantagenet's facility is undersized for its current population and flow rates.

Table 8-1 Current Lagoon Assessment

Municipality	Design Flow (m^3/d)	Volume (1000 m^3)	Flow 1991 (m^3/d)	Volume Required (1000 m^3)
Casselman	1,346	243	1,000	
Chesterville	1,040	185	811	
Embrun	1,798	800	1,300	
Maxville	450	164	164	
Plantagenet	590	100	617	9.9
Russell	1,000	235	529	
Spencerville	318	116	140	
St. Isidore	655	220	348	
Williamsburg	1,220	75	176	
Winchester	1,725	470	1,725	
Industry				
Ault	573	194	531	
Nestles	2,045	226	619	

The treatment upgrade options considered include:

- Lagoon Facility
- Extended Aeration with Nitrification Facility
- New Hamburg Process Facility
- Piping Effluent to an Alternate Location

Infiltration treatment facilities were not considered feasible in the South Nation watershed due to the high groundwater table in this region.

In this section, estimated capital and operating costs for the various treatment options for future 7Q20 flow conditions are shown in Tables 8-2 to 8-9. Annual cost comparisons for different treatment options for both present and future conditions and for 7Q20 and 30Q20 flows are presented in Tables 8-10 to 8-13. Complete capital and operating costs tables for the existing 7Q20 and 30Q20 flows and for future 30Q20 flow conditions can be found in Appendix I.

It is important to note that in developing cost estimates for different treatment options, the cost of land, metering/sampling equipment, major maintenance and synthetic liners have not been included. The need to install liners within lagoon cells, post aeration cells, and/or sand filters beds will be determined on a site specific basis depending on MOE's "Reason Use Policy". The costs involved in providing such liners are significant and may noticeably augment the capital cost estimates presented herein.

The treatment costs for the St. Albert Cheese factory were not estimated because the wastewater generated from this industrial facility is likely to contain a high concentration of pollutants and therefore require a custom designed wastewater treatment facility.

8.1 LAGOON FACILITIES

The lagoon volume upgrading requirements to meet the future population estimates are shown in Table 8-2. The treatment systems were sized for discharge on an annual basis i.e. 12 months of storage capacity is required (which is conservative). The following design criteria were assumed for the lagoon facilities (MOE, 1984a):

- loading $\leq$ 22 kg BOD/ha/d
- maximum lagoon size of approximately 8 ha, 4 ha preferred
- 0.6 m freeboard
- 2.4 m minimum berm width
- 3.0 m between bedrock and cell bottom
- cell bottom must be 1.2 m above high groundwater table
- maximum liquid depth of 1.8 m

The following assumptions regarding lagoon construction and site conditions were made:

- construct berms with native compacted soil
- side slopes 4:1 inside, 3:1 outside
- access ramp provided for each lagoon
- outside of berms seeded with grass

The capital cost breakdown for the Bourget lagoon facility with an annual volume of 230,200 m^3 is as follows:

excavation and berm construction	$395,000
surplus material removal	$20,000
fencing and gates	$65,000
piping and valves	$225,000
gravel roadways	$90,000
lighting and power	$40,000
phosphorus removal equipment	$40,000
insulated steel service building	$40,000
Total	$915,000

Capital costs for the other municipalities were extrapolated using the function presented below which relates capital costs of different sized facilities (Peters and Timmerhaus, 1980):

$$C_n = C(R)^x \quad \textbf{(8-1)}$$

where C_n is the cost of the new facility
C is the cost of the original facility
R is the ratio of the flow (or volume) for the new facility to the original facility
x is recommended to be 0.6 - 0.7. A value of 0.6 was used.

Spring discharge of lagoon contents will necessitate a post-aeration cell for hydrogen sulphide control since the optimum spring discharge window is expected to partially occur in March and April when the lagoon may still be partially covered with ice (see Sec. 3.3.3). The post-aeration cell design parameters for H_2S stripping include:

- annual discharge over 30 days
- 24 h detention time
- water depth of 3.5 m
- 0.6 m freeboard
- aeration requirement of Q = 0.33 $L/m^2/s$
- assumed 70% efficiency on equipment

The capital costs involved in providing a post aeration cell for the Bourget lagoon treatment facility was developed as follows. Since the post aeration cell is basically a lagoon with aeration equipment, the cell costs were determined using Eq. 8-1. The cell size for a 30 day discharge and 24 hour retention is equal to 230,200 m³/30 or 7,673 m³. Therefore,

cell ($915,000)[7,673/230,200]$^{0.6}$	$118,900
aeration equipment	$225,000
total	$343,900

Capital costs for the other municipalities were extrapolated using Eq. 8-1. Table 8-2 presents the capital costs for each community to upgrade the existing lagoon facilities, install new lagoon facilities and provide post-aeration.

8.2 EXTENDED AERATION WITH NITRIFICATION FACILITY

Extended aeration facility design requirements to meet future population estimates are shown in Table 8.3 Since the receiving streamflows are negligible during the summer months, a storage lagoon is required to collect the treated wastewater until the discharge can occur. The holding lagoon was sized for a storage capacity of 12 months (which is conservative). For municipalities with existing lagoon facilities, the lagoon can serve as the holding lagoon and extra storage will be provided.

Capital cost estimates for the extended aeration with nitrification facilities were based on "package" type plants completely installed. The costs were determined for two ranges of flows; $1,760/m³/d for flows below 400 m³/d and $1,540/m³/d for flows greater than 400 m³/d. For Bourget, the extended aeration facility costs are $1,540/m³/d (630.7 m³/d) or $971,300.

Cost estimates for holding lagoons are as presented in Section 8.1. The capital costs to install an extended aeration facility and holding lagoon are shown in Table 8-3.

8.3 NEW HAMBURG PROCESS

The New Hamburg process consists of a lagoon facility with slow sand filtration for effluent polishing. The slow sand filtration system also known as an intermittent sand filtration system was sized using a design load of 4,500m³/ha/d and allowing at least a 2 day recovery period, i.e. 3 filters required; 1 in use while 2 are recovering.

The capital cost for the sand filtration beds was established using the following relation:

$$C = \$200(L) + \$175{,}000 \qquad (8\text{-}2)$$

Where L is the load in m³/d to the filtration system.

This cost includes the berm construction, filter media, pumping station, piping and drains and it excludes land costs and liners. This cost was determined based on an 8 month filtration period as currently being used at the New Hamburg treatment facility. For example for the Bourget facility, the cost of the filter is as follows;
($200) 230,200m³/[8mo • 30.5d] + $175,000 = $363,700.

Cost estimates for the holding lagoons are as presented in Section 8.1. Capital costs to install a filtration system and holding lagoon are shown in Table 8-4.

8.4 PIPING EFFLUENT TO AN ALTERNATE LOCATION

The annual costs for piping effluent to an alternate location are shown in Table 8-6. In the case where insufficient dilution capacity exists to meet the PWQOs, capital cost estimates were established to pump the treated wastewater from the lagoon facility during the optimum window to a point where sufficient dilution capacity exists.

A capital cost estimate of $100,000 was assumed for each pumping station. Capital cost estimates for effluent piping are presented in Table 8-6.

8.5 ANNUAL COSTS

The annual costs for the various options including capital and operating costs using an amortization of 20 years and 10% discount rate are given in Tables 8-6 to 8-9. It was assumed that the operating cost of the New Hamburg process would be the same as the operating cost of a extended aeration system.

Annual cost of the lagoon upgrading with post aeration cell is somewhat less than the other treatment options presented herein since it is the least sophisticated and the least demanding in terms of maintenance. There is generally not a great variance between costs for the extended aeration treatment and the New Hamburg process, the New Hamburg being the least expensive of the two.

8.5.1 Cost Estimates for Present and Future Allocation Solutions

Annual cost estimates and the least cost option for the optimum solutions given in Chapter 6 for the present and future conditions are given in Tables 8-10 to 8-13. Cost estimates of the upgrades have been developed in the same manner as the above costs and may be found in Appendix I.

Optimum solutions were focused on the minimum degree of treatment required at each location to maintain PWQO standards in the receiving body of water. If a treatment process produces a better quality effluent than the minimum treatment required at a location it becomes the treatment of choice. The stream condition is not impaired by the installation of a more advanced process which produces better effluent quality.

In general, the New Hamburg process is the least costly of the advanced options. This is due to the assumptions that existing and new lagoons will not require a liner, existing lagoons may be used for storage (i.e. no modifications required), and only

filters need to be added. If the lagoons and filter beds (NHP) require a liner, then the New Hamburg process will become more costly (in general) than the extended aeration process.

Table 8-2: Lagoon Upgrading Requirements and Associated Costs

Municipality	C of A		Future Flow Year 2011		Current Lagoon Volume (m3)	Additional Requirements		Num./Size Additional Lagoons (#/ha)	Post Aeration Cell (ha) [5]	Estimated Lagoon Cost ($) [6]	Estimated Aeration Cell Cost ($)
	Design Flow (m3/d)	Discharge Frequency [2]	Daily (m3/d)	Annual (m3)		Storage Volume (m3)	Area d = 1.5m (ha)				
Bourget [1]			631	230,200	0	230,200	15.3	2/3.83,1/7.65	0.22	915,000	343,900
Casselman	1,346	SA	2,587	944,300	243,000 [3]	701,300	46.8	6/7.8	0.90	1,785,300	802,100
Chesterville	1,040	SA	873	318,800	185,000 [3]	133,800	8.9	2/4.45	0.30	660,700	418,100
Crysler [1]			600	219,000	0	219,000	14.6	2/3.65,1/7.3	0.21	888,000	333,800
Embrun	1,798	A	4,250	1,551,100	800,000 [3]	751,100	50.1	6/8.35	1.48	1,860,300	1,080,300
Fournier [1]			139	50,900	0	50,900	3.4	2/0.85,1/1.7	0.05	370,000	139,100
Maxville	450	A	168	61,300	164,000 [4]	0	0.0		0.06	0	155,500
Moose Creek [1]			278	101,600	0	101,600	6.8	2/1.7,1/3.4	0.10	560,100	210,500
Plantagenet	590	SA	696	254,100	100,000 [3]	154,100	10.3	2/5.35	0.24	719,200	364,900
Russell	1,000	A	2,371	865,300	235,000 [3]	630,300	42.0	6/7	0.82	1,674,500	761,100
Spencerville			190	69,400	116,000 [4]	0	0.0		0.07	0	167,500
St. Albert [1]			379	138,400	0	138,400	9.2	2/2.3,1/4.6	0.13	674,300	253,400
St. Isidore	655	A	484	176,700	220,000 [3]	0	0.0		0.17	0	293,400
Williamsburg	1,220	SA	194	70,800	75,000 [3]	0	0.0		0.07	0	169,500
Winchester	1,725	SA	1,965	717,200	470,000 [3]	247,200	16.5	2/8.25	0.68	955,000	680,100

Notes:

1. Proposed future WPCP
2. SA - semi-annual discharge; A - annual discharge
3. Based on 1.5 m depth of liquid discharged and 0.30 m retained
4. C of A volume (assumed to be the active volume)
5. Based on annual discharge over 30 days and retention time of 24 hours.
6. Costs are given for lagoon discharged on an annual basis.
   Lagoon costs take into account existing lagoon volume.

   Costs are for evaluation purposes only and must not be used for budget planning

Table 8-3: Treatment Plant Facility Design and Associated Costs
Extended Aeration with Nitrification

Municipality	Future Pop. 2011	Mass of BOD-in F [2] (kg BOD/d)	Aeration Tank				Solids Loading Rate [7] (kg SS/d)	Clarifier Cost [8] ($)	Storage Lagoon Est. Cost [9] ($)
			Required Volume [3] (m3)	HRT [4] (h)	Requirements				
					Oxygen [5] (kg O2/h)	Power [6] (kW)			
Bourget [1]	1,577	118	493	19	11.6	23.2	5,045	971,300	915,000
Casselman	6,368	478	1,990	18	46.9	93.9	20,697	3,984,200	1,785,300
Chesterville	1,656	124	518	15	12.2	24.4	6,987	1,345,100	660,700
Crysler [1]	1,500	113	469	19	11.1	22.1	4,800	924,000	888,000
Embrun	10,874	816	3,398	19	80.2	160.3	33,997	6,544,400	1,860,300
Fournier [1]	349	26	109	19	2.6	5.1	1,116	245,400	370,000
Maxville	810	61	253	36	6.0	11.9	1,344	295,600	0
Moose Creek [1]	696	52	218	19	5.1	10.3	2,227	489,900	560,100
Plantagenet	1,098	82	343	15	8.1	16.2	5,569	1,072,100	719,200
Russell	6,404	480	2,001	20	47.2	94.4	18,965	3,650,900	1,674,500
Spencerville	615	46	192	24	4.5	9.1	1,521	334,600	0
St. Albert [1]	948	71	296	19	7.0	14.0	3,033	667,400	674,300
St. Isidore	1,040	78	325	16	7.7	15.3	3,873	745,500	0
Williamsburg	485	36	152	19	3.6	7.1	1,552	341,400	0
Winchester	2,642	198	826	15	19.5	38.9	15,719	3,026,000	955,000

Notes:

1. Proposed future WPCP
2. Design BOD loading = 75 g/cap.d
3. Based on MOE guideline OLR = 0.24kg BOD/m3/d
4. MOE guideline, minimum HRT = 15 h
5. MOE guideline based on applied load; oxygen demand = 1.5 kg O2/kg BOD + 4.6 kg O2/kg TKN
   Design TKN loading = 0.014 kg/cap.d
6. Assume efficiency of 0.5 kg O2/kWh for coarse bubble aeration
7. Assume MLVSS conc = 3000 mg/L = 3 kg/m3 and MLVSS/MLSS = 0.75. Flow peaking factor of 2.0.
8. Costs are based on $1,760/m3/d for a facility with a design flow below 400 m3/d and $1,540/m3/d over 400 m3/d.
9. Costs are given for lagoon discharged on an annual basis.

   As per MOE guideline, set F/Mv = 0.10/d; Use square concrete tank.
   Costs are for evaluation purposes only and must not be used for budget planning

Table 8-4: New Hamburg Process Cost Estimates

Municipality	Future Flow Year 2011		New Hamburg Sand Filter Est. Costs [2] ($)	Storage Lagoon Est. Cost [3] ($)
	Daily (m3/d)	Annual (m3)		
Bourget [1]	631	230,200	363,700	915,000
Casselman	2,587	944,300	949,000	1,785,300
Chesterville	873	318,800	436,300	660,700
Crysler [1]	600	219,000	354,500	888,000
Embrun	4,250	1,551,100	1,446,400	1,860,300
Fournier [1]	139	50,900	216,700	370,000
Maxville	168	61,300	225,200	0
Moose Creek [1]	278	101,600	258,300	560,100
Plantagenet	696	254,100	383,300	719,200
Russell	2,371	865,300	884,300	1,674,500
Spencerville	190	69,400	231,900	0
St. Albert [1]	379	138,400	288,400	674,300
St. Isidore	484	176,700	319,800	0
Williamsburg	194	70,800	233,000	0
Winchester	1,965	717,200	762,900	955,000

Notes:

1. Proposed future WPCP
2. Assuming annual flow is filtered over a 8 month period.
   Design loading of 4500 m3/ha/d.
   Application period of 3 hours.
   Total of 3 filters allowing a 2 day recovery period.
3. Costs are given for lagoon discharged on an annual basis.

   Costs are for evaluation purposes only and must not be used for budget planning.

Table 8-5: Annual Operating Costs for Treatment Options

Municipality	Phosphorus Removal Chemicals [2] ($)	Lagoon Facilities		Ext. Aeration w/ Nitrification			New Hamburg	
		Manpower [3] ($)	Total ($)	Power [4] ($)	Manpower [5] ($)	Total ($)	Manpower [5] ($)	Total ($)
Bourget [1]	5,200	10,400	15,600	10,200	52,000	67,400	52,000	57,200
Casselman	21,200	10,400	31,600	41,100	52,000	114,300	52,000	73,200
Chesterville	7,200	10,400	17,600	10,700	52,000	69,900	52,000	59,200
Crysler [1]	4,900	10,400	15,300	9,700	52,000	66,600	52,000	56,900
Embrun	34,900	10,400	45,300	70,200	52,000	157,100	52,000	86,900
Fournier [1]	1,100	10,400	11,500	2,300	52,000	55,400	52,000	53,100
Maxville	1,400	10,400	11,800	5,200	52,000	58,600	52,000	53,400
Moose Creek [1]	2,300	10,400	12,700	4,500	52,000	58,800	52,000	54,300
Plantagenet	5,700	10,400	16,100	7,100	52,000	64,800	52,000	57,700
Russell	19,500	10,400	29,900	41,300	52,000	112,800	52,000	71,500
Spencerville	1,600	10,400	12,000	4,000	52,000	57,600	52,000	53,600
St. Albert [1]	3,100	10,400	13,500	6,100	52,000	61,200	52,000	55,100
St. Isidore	4,000	10,400	14,400	6,700	52,000	62,700	52,000	56,000
Williamsburg	1,600	10,400	12,000	3,100	52,000	56,700	52,000	53,600
Winchester	16,100	10,400	26,500	17,100	52,000	85,200	52,000	68,100

Notes:

1. Proposed future WPCP
2. Based on dose of 100 mg/L and liquid alum cost of $225/tonne (delivery cost included)
3. Based on 1/5 personyear/year and personyear cost of $52,000
4. Based on $0.05/kWh and 24 h operating time
5. Based on 1 personyear/year and personyear cost of $52,000

Excludes any major maintenance costs.
Costs are for evaluation purposes only and must not be used for budget planning

Table 8-6: Relocation of Effluent Discharge Points

Source of Wastewater			Ault	Maxville	Winchester	Embrum
Discharge Location			East Castor	West Scotch	S.N. River	S.N. River
Discharge Node			AB	T	N	J
Average Daily Discharge Flow [1]		(m3/d)	30,333	4,087	21,733	47,003
Calculated	Maximum Pipe Diameter [2]	(mm)	705	259	597	877
	Minimum Pipe Diameter [3]	(mm)	386	142	327	481
Chosen Pipe Diameter		(mm)	510	203	406	610
Distance between Discharge Points [4]		(km)	3.0	3.0	6.0	15.0
Forcemain Cost Estimate [5]		($)	1,530,000	609,000	2,436,000	9,150,000
Pumping Station Costs		($)	100,000	100,000	100,000	100,000
Annual Cost Estimate [6]		($)	191,500	83,300	297,900	1,086,500

Notes:

1. Discharged over 15 days for Ault and Maxville and 33 days for Winchester and Embrum.
2. Based on a minimum velocity of 0.9 m/s
3. Based on a maximum velocity of 3.0 m/s
4. Pipe distances are based on straight line distance with no rock deposits or other obstructions or complicating factors such as rises in the pipeline. Pumping station costs are not included.
5. Assumed a pipe cost of $1/mm/m installed
6. Amortized over 20 year at a discount rate of 10%.

Costs are for evaluation purposes only and must not be used for budget planning

Table 8-7: Annual Cost Estimates for Lagoon Upgrading with Post Aeration Cell [2]

Municipality	Capital Costs			Operating Costs	Total Yearly Costs
	Lagoon ($)	AE Cell ($)	Total ($)	($)	($)
Bourget [1]	915,000	343,900	1,258,900	15,600	163,500
Casselman	1,785,300	802,100	2,587,400	31,600	335,500
Chesterville	660,700	418,100	1,078,800	17,600	144,300
Crysler [1]	888,000	333,800	1,221,800	15,300	158,800
Embrun	1,860,300	1,080,300	2,940,600	45,300	390,700
Fournier [1]	370,000	139,100	509,100	11,500	71,300
Maxville	0	155,500	155,500	11,800	30,100
Moose Creek [1]	560,100	210,500	770,600	12,700	103,200
Plantagenet	719,200	364,900	1,084,100	16,100	143,400
Russell	1,674,500	761,100	2,435,600	29,900	316,000
Spencerville	0	167,500	167,500	12,000	31,700
St. Albert [1]	674,300	253,400	927,700	13,500	122,500
St. Isidore	0	293,400	293,400	14,400	48,900
Williamsburg	0	169,500	169,500	12,000	31,900
Winchester	955,000	680,100	1,635,100	26,500	218,600

Notes:

1. Proposed future WPCP
2. Amortized over 20 year at a discount rate of 10%.

Costs are for evaluation purposes only and must not be used for Budget planning.

Table 8-8: Annual Cost Estimates for Extended Aeration Treatment with Lagoon Storage[2]

Municipality	Capital Costs			Operating Costs	Total Yearly Costs
	Lagoon ($)	Plant ($)	Total ($)	($)	($)
Bourget [1]	915,000	971,300	1,886,300	67,400	289,000
Casselman	1,785,300	3,984,200	5,769,500	114,300	792,000
Chesterville	660,700	1,345,100	2,005,800	69,900	305,500
Crysler [1]	888,000	924,000	1,812,000	66,600	279,400
Embrun	1,860,300	6,544,400	8,404,700	157,100	1,144,300
Fournier [1]	370,000	245,400	615,400	55,400	127,700
Maxville	0	295,600	295,600	58,600	93,300
Moose Creek [1]	560,100	489,900	1,050,000	58,800	182,100
Plantagenet	719,200	1,072,100	1,791,300	64,800	275,200
Russell	1,674,500	3,650,900	5,325,400	112,800	738,300
Spencerville	0	334,600	334,600	57,600	96,900
St. Albert [1]	674,300	667,400	1,341,700	61,200	218,800
St. Isidore	0	745,500	745,500	62,700	150,300
Williamsburg	0	341,400	341,400	56,700	96,800
Winchester	955,000	3,026,000	3,981,000	85,200	552,800

Notes:

1. Proposed future WPCP
2. Amortized over 20 year at a discount rate of 10%.

Costs are for evaluation purposes only and must not be used for Budget planning.

Table 8-9: Annual Cost Estimates for the New Hamburg Process with Lagoon Storage[2]

Municipality	Capital Costs			Operating Costs	Total Yearly Costs
	Lagoon ($)	NH Filter ($)	Total ($)	($)	($)
Bourget [1]	915,000	363,700	1,278,700	57,200	207,400
Casselman	1,785,300	949,000	2,734,300	73,200	394,400
Chesterville	660,700	436,300	1,097,000	59,200	188,100
Crysler [1]	888,000	354,500	1,242,500	56,900	202,800
Embrun	1,860,300	1,446,400	3,306,700	86,900	475,300
Fournier [1]	370,000	216,700	586,700	53,100	122,000
Maxville	0	225,200	225,200	53,400	79,900
Moose Creek [1]	560,100	258,300	818,400	54,300	150,400
Plantagenet	719,200	383,300	1,102,500	57,700	187,200
Russell	1,674,500	884,300	2,558,800	71,500	372,100
Spencerville	0	231,900	231,900	53,600	80,800
St. Albert [1]	674,300	288,400	962,700	55,100	168,200
St. Isidore	0	319,800	319,800	56,000	93,600
Williamsburg	0	233,000	233,000	53,600	81,000
Winchester	955,000	762,900	1,717,900	68,100	269,900

Notes:

1. Proposed future WPCP
2. Amortized over 20 year at a discount rate of 10%.

Costs are for evaluation purposes only and must not be used for Budget planning.

Table 8-10: Annual Cost Estimates of Different Treatment Options

Municipality	Option A Lagoon ($)	Option B Piping ($)	Option C Extended Aeration ($)	Option D New Hamburg ($)	Acceptable Option or Combination of Options	Least Cost Acceptable Option or Combination of Options	Minimum Cost ($)
Bourget [1]	163,500		289,000	207,400	A, C, D	A	163,500
Casselman	335,500		792,000	394,400	A, C, D	A	335,500
Chesterville	144,300		305,500	188,100	A, C, D	A	144,300
Crysler [1]	158,800		279,400	202,800	A, C, D	A	158,800
Embrun	390,700	1,086,500	1,144,300	475,300	C, D, A+B, C+B, D+B	D	475,300
Fournier [1]	71,300		127,700	122,000	A, C, D	A	71,300
Maxville	30,100	83,300	93,300	79,900	A+B, C+B, D+B	A+B	113,400
Moose Creek [1]	103,200		182,100	150,400	C, D	D	150,400
Plantagenet	143,400		275,200	187,200	A, C, D	A	143,400
Russell	316,000		738,300	372,100	A, C, D	A	316,000
Spencerville	31,700		96,900	80,800	A, C, D	A	31,700
St. Albert [1]	122,500		218,800	168,200	A, C, D	A	122,500
St. Isidore	48,900		150,300	93,600	A, C, D	A	48,900
Williamsburg	31,900		96,800	81,000	C, D	D	81,000
Winchester	218,600	297,900	552,800	269,900	A+B, C+B, D+B	A+B	516,500
Ault		191,500			B	B	191,500
Nestle							
St. Albert Cheese							

Notes: Costs are for evaluation purposes only and must not be used for budget planning

Table 8-11: Annual Cost Estimates of Different Treatment Options

Municipality	Option A Lagoon ($)	Option B Piping ($)	Option C Extended Aeration ($)	Option D New Hamburg ($)	Acceptable Option or Combination of Options	Least Cost Acceptable Option or Combination of Options	Minimum Cost ($)
Casselman	145,300		330,000	189,300	A, C, D	A	145,300
Chesterville	133,500		284,500	177,100	A, C, D	A	133,500
Embrun	83,400	637,200	320,500	128,900	A, C, D, A+B, C+B, D+B	A	83,400
Maxville	29,700	82,200	92,400	79,600	A+B, C+B, D+B	A+B	111,900
Plantagenet	129,900		249,100	173,900	A, C, D	A	129,900
Russell	51,100		163,600	95,400	A, C, D	A	51,100
Spencerville	27,900		84,300	78,600	A, C, D	A	27,900
St. Isidore	41,600		131,300	87,700	A, C, D	A	41,600
Williamsburg	30,600		92,500	80,100	C, D	D	80,100
Winchester	184,700	300,700	478,700	233,700	A+B, C+B, D+B	A+B	485,400
Ault							
Nestle							
St. Albert Cheese							

Notes:

Costs are for evaluation purposes only and must not be used for budget planning
Refer to Appendix I for tables with present 7Q20 flows.

Table 8-12: Annual Cost Estimates of Different Treatment Options

Municipality	Option A Lagoon ($)	Option B Piping ($)	Option C Extended Aeration ($)	Option D New Hamburg ($)	Acceptable Option or Combination of Options	Least Cost Acceptable Option or Combination of Options	Minimum Cost ($)
Bourget [1]	163,500		289,000	207,400	A, C, D	A	163,500
Casselman	335,500		792,000	394,400	A, C, D	A	335,500
Chesterville	144,300		305,500	188,100	A, C, D	A	144,300
Crysler [1]	158,800		279,400	202,800	A, C, D	A	158,800
Embrun	390,700	1,086,500	1,144,300	475,300	A, C, D, A+B, C+B, D+B	A	390,700
Fournier [1]	71,300		127,700	122,000	A, C, D	A	71,300
Maxville	30,100	64,600	93,300	79,900	A+B, C+B, D+B	A+B	94,700
Moose Creek [1]	103,200		182,100	150,400	A, C, D	A	103,200
Plantagenet	143,400		275,200	187,200	A, C, D	A	143,400
Russell	316,000		738,300	372,100	A, C, D	A	316,000
Spencerville	31,700		96,900	80,800	A, C, D	A	31,700
St. Albert [1]	122,500		218,800	168,200	A, C, D	A	122,500
St. Isidore	48,900		150,300	93,600	A, C, D	A	48,900
Williamsburg	31,900		96,800	81,000	C, D	D	81,000
Winchester	218,600	226,700	552,800	269,900	A+B, C+B, D+B	A+B	445,300
Ault		119,200			B	B	119,200
Nestle							
St. Albert Cheese							

Notes: Costs are for evaluation purposes only and must not be used for budget planning
Refer to Appendix I for tables with future 30Q20 flows.

Table 8-13: Annual Cost Estimates of Different Treatment Options

Municipality	Option A Lagoon ($)	Option B Piping ($)	Option C Extended Aeration ($)	Option D New Hamburg ($)	Acceptable Option or Combination of Options	Least Cost Acceptable Option or Combination of Options	Minimum Cost ($)
Casselman	145,300		330,000	189,300	A, C, D	A	145,300
Chesterville	133,500		284,500	177,100	A, C, D	A	133,500
Embrun	83,400	549,100	320,500	128,900	A, C, D, A+B, C+B, D+B	A	83,400
Maxville	29,700	47,000	92,400	79,600	A+B, C+B, D+B	A+B	76,700
Plantagenet	129,900		249,100	173,900	A, C, D	A	129,900
Russell	51,100		163,600	95,400	A, C, D	A	51,100
Spencerville	27,900		84,300	78,600	A, C, D	A	27,900
St. Isidore	41,600		131,300	87,700	A, C, D	A	41,600
Williamsburg	30,600		92,500	80,100	C, D	D	80,100
Winchester	184,700	156,200	478,700	233,700	A+B, C+B, D+B	A+B	340,900
Ault		82,200			B	B	82,200
Nestle							
St. Albert Cheese							

Notes: Costs are for evaluation purposes only and must not be used for budget planning
Refer to Appendix I for tables with present 30Q20 flows.

SECTION 9

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS AND RECOMMENDATIONS

9.1 CONCLUSIONS

The following conclusions can be drawn from this study.

- Water quality in the South Nation River System has been improving during the past 12 years. Installation of sewerage and wastewater treatment capacity has probably been a factor.

- Phosphorus loads into the receiving water have not changed during the past 12 years and instream phosphorus concentrations consistently exceed the PWQO.

- Non-point source contributions of phosphorus and nitrogen constitute more than 93 and 98% respectively of the loads of these nutrients in the streams on an annual basis. However, during periods when point sources are discharging, point sources have been observed to comprise more than 50% of the total ammonia load in the streams.

- Lagoon effluent must be treated with a post aeration cell to insure that instream hydrogen sulphide standards are not violated. Lagoon effluent must also be treated with a post aeration cell to insure that the in-pipe non-lethality standard for H_2S is not violated.

- Lagoon effluents are likely to violate the in-pipe lethality ammonia standard during spring discharge without extraordinary measures.

- Extended aeration and New Hamburg processes produce effluent that is not likely to violate the effluent lethality criteria. Storage of effluent from these processes in a lagoon for seasonal or annual discharge will likely result in algal blooms that will periodically raise effluent pH to values that will cause a lethality violation for un-ionized ammonia.

The optimizations for wastewater discharge and treatment measures were carried out on the basis of minimizing both the amount of treatment and the distance of relocation of effluent discharge points. This does not ensure a uniform distribution of costs to all point sources, but minimizes the costs for the watershed as a whole.

- Ammonia was the limiting parameter for effluent discharge.

- Dissolved oxygen falls below PWQO in the fall even if no wastewater is released.

Under conditions of 30Q20 streamflow, with constant effluent discharge rates, and minimum stream dilution volumes defined by the windows approach developed in this

report the conclusions below are reached. The windows were based on a moving average of 30Q20 flows during the spring and fall seasons.

At Present Conditions

- The communities of Winchester and Maxville and the industry of Ault on tributaries to the South Nation River must relocate their effluent discharge points. Winchester must route its effluent to the South Nation River. Maxville and Ault must move their discharge location further downstream in the streams into which they discharge. Williamsburg must upgrade treatment to extended aeration or better.

- All other communities are able to use lagoon treatment.

- All point sources are able to discharge during the maximum window.

At Future Conditions

- The communities of Winchester and Maxville and the industry of Ault on tributaries to the South Nation River must relocate their effluent discharge points. Winchester must route its effluent to the South Nation River. Maxville and Ault must move their discharge location further downstream in the streams into which they discharge. Williamsburg must upgrade treatment to extended aeration or better.

- All other communities are able to use lagoon treatment.

- A staged discharge scheme has been developed for the maximum and average windows and is required to meet PWQO standards.

Under conditions of 7Q20 streamflow, with constant effluent discharge rates, and minimum stream dilution volumes defined by the windows approach developed in this report, the conclusions below are reached. The windows were based on a moving average of 7Q20 flows during the spring and fall seasons.

At Present Conditions

- The communities of Winchester and Maxville and the industry of Ault on tributaries to the South Nation River must relocate their effluent discharge points. Winchester must route its effluent to the South Nation River. Maxville and Ault must move their discharge location further downstream in the streams into which they discharge. Williamsburg must upgrade its treatment to extended aeration or better.

- All other communities are able to use lagoon treatment.

- A staged discharge scheme over the three windows has been developed to meet PWQO.

At Future Conditions

- The communities of Winchester and Maxville and the industry of Ault on tributaries to the South Nation River must relocate their effluent discharge points. Winchester must route its effluent to the South Nation River. Maxville and Ault must move their discharge location further downstream in the streams into which they discharge. In addition to Williamsburg, the communities of Embrun and Moose Creek must upgrade treatment to extended aeration or better.

- All other communities are able to use lagoon treatment.

- A staged discharge scheme has been developed utilizing all three windows and is required to meet PWQO standards.

The 75% zone of passage objective can be achieved everywhere on the South Nation River for both present and future 30Q20 conditions, and for the present 7Q20 conditions. It cannot be achieved on the upper reaches of the South Nation River for future 7Q20 conditions. Most tributary streams and drains cannot achieve the 75% zone of passage objective under any flow conditions.

9.2 RECOMMENDATIONS

- Discrepancies in discharge criteria in the MOE Sewage Compliance Status Report and Certificates of Approval should be resolved.

- Extended aeration activated sludge is the most suitable mechanical wastewater treatment option for small communities. This treatment process will not generally produce hydrogen sulphide.

- Communities with sewerage flowrates in excess of 500 L/cap/d should examine their sewer systems and undertake remedial measures to reduce flowrates.

- This study should be extended to examine effluent discharge during other months of the year, particularly where extended aeration and New Hamburg processes are indicated, to more fully utilize the assimilative capacity of the streams.

- A stochastic analysis of the system would provide a valuable addition to understanding the frequency of PWQO violations.

- Real time control of effluent discharges with variable discharge rates contingent on streamflow offers the opportunity to reduce the treatment upgrades and distances required for effluent point relocation. A study of variable rate discharge should be commissioned to evaluate costs and advantages of this approach.

- Opportunities for increasing the assimilative capacity of the river through flow augmentation could be explored.

- More extensive data collection, particularly for water quality parameters, in the South Nation River system is required to improve analysis and forecasts of system behaviour.

- A study of non-point sources of pollution, particularly with regard to nutrients is required to determine the extent to which stream quality could be improved with corrective measures. Non-point sources often contribute more than 50% of the ammonia load that the streams are able to assimilate during critical months when point sources are discharging.

- Wastewater allocation can be accomplished according to many different paradigms. Unused stream capacity is a function of background conditions and upstream inputs. Various allocation rules should be examined to prioritize them on a physical basis. Social, economic, environmental and aesthetic values and goals should be examined in parallel to formulate a consistent policy on wastewater allocation.

  Specifically different population allocation schemes should be examined (geometric growth, arithmetic growth, etc.) along with the economic impacts of the various growth schemes.

  The impacts of various stream standards on wastewater treatment and allocation should be examined.

SECTION 10

REFERENCES

REFERENCES

Acres International Ltd. (1990). "St. Albert Cheese Wastewater Treatment System", report for St. Albert Cooperative Cheese Manufacturing Association, P.O. Box 30, St. Albert, Ont.

Briggs, T., (1990). "Rural Beaches Strategy Program, 1989 Progress Report", Upper Thames River Conservation Authority, (May).

Brown, L.C. and T.O. Barnwell, Jr. (1987). "The Enhanced Stream Water Quality Models QUAL2E and QUAL2E-UNCAS: Documentation and User Manual", Environmental Research Laboratory, USEPA, Athens, GA.

Condie, R. and L.C. Cheng (1983). "Low Flow Frequency Analysis Program LOWFLOW" Inland Waters Directorate, Environment Canada, Ottawa.

Churchill, M., H. Elmore, and R. Buckingham (1962). "The Prediction of Stream Reaeration Rates", *J. of the Sanitary Engineering Division*, ASCE, 88, SA4, pp. 1-46.

Covar, A.P. (1976). "Selecting the Proper Reaeration Coefficient for Use in Water Quality Models", presented at the USEPA Conference on Environmental Simulation and Modelling.

Cumming Cockburn Limited (1989). "Low Flow Characteristics in Ontario", report for Water Resources Branch, Ontario Ministry of the Environment, Toronto.

Doneker, R.L. and Jirka, G.H. (1990) Expert System for Hydrodynamic Mixing Zone Analysis of Conventional and Toxic Submerged Single Port Discharges (CORMIX1), Report No. EPA/600/3-90/012, U.S. Environmental Protection Agency, Env. Research Laboratory, Athens, Georgia.

Environment Canada (1985). *Cold Climate Sewage Lagoons*, Proceedings of the June 1985 Workshop in Winnipeg, Manitoba, Environment Canada, Ottawa.Ott, W.R. (1978). *Environmental Indices: Theory and Practice*, Ann Arbor Science Publishers, Inc., Ann Arbor, MI.

Hayman, D.G., (1986). "An Evaluation Of The Effect Of Soil Conservation Practices On Water Quality In A Demonstration Versus Control Subwatershed Study In The Avon River", Upper Thames River Conservation Authority.

Hayman, D. G. (1988). "A Clean Up Rural Beaches Plan (CURB) For Fanshawe, Pittock and Wildwood Reservoir in the Upper Thames River Conservation Authority Watershed," Report prepared by the UTRCA for the Ontario Ministry of the Environment, Toronto.

Hayman, D.G. (1992). personal communication, Rural Water Quality Program Coordinator, Upper Thames River Conservation Authority, London, Ont.

Kostuch Engineering Ltd. (1990a). "Preliminary Report, Village of Fournier, Township of South Plantagenet", MOE Project No. 3-0786, Ontario Ministry of the Environment.

Kostuch Engineering Ltd. (1990b). "Village of Crysler, Communal Sewage and Water Servicing Environmental Study Report", Vol. 1, Environmental Study Report, South Nation River Conservation Authority.

MacLaren Plansearch (1982). "Water Resources Study Component of the South Nation River Basin", report submitted to South Nation River Conservation Authority.

Metcalf & Eddy, Inc. (1991). *Wastewater Engineering: Treatment, Disposal and Reuse*, McGraw-Hill, Toronto.

MOE (1980). "Winter Lagoon Data for Southwestern Ontario, Winter 1978-79", Ontario Ministry of the Environment, Toronto.

MOE (1984a). *Guidelines for the Design of Sewage Treatment Works*, Ontario Ministry of the Environment, Toronto.

MOE (1984b). *Water Management: Goals, Policies, Objectives and Implementation Procedures of the Ministry of the Environment*, Ontario Ministry of the Environment, Toronto.

MOE (1986-89). *Reports on Discharges from Municipal and Industrial Sewage Treatment Plants in Ontario*, Ontario Ministry of the Environment, Toronto.

MOE (1988). "Performance Predictions for Wastewater Treatment Lagoon Facilities in Ontario", Water Resources Branch, Ontario Ministry of the Environment, Toronto.

MOE (1990a). *Sewage Compliance Status Report*, Ontario Ministry of the Environment, Toronto.

MOE (1990b). "Winter Lagoon Data for Southwestern Ontario", Ontario Ministry of the Environment Water Resources Assessment Unit, Southeastern Region.

MOE (1991a). "Nitrogen Data Set for Lagoon Facilities in the South Nation Watershed", Ontario Ministry of the Environment, Toronto.

MOE (1991b). "South Nation River Stations Sample Information System Data Set", Ontario Ministry of the Environment, Toronto.

MOE (1992). A. Bacchus, Water Resources Branch, Ontario Ministry of the Environment, Toronto.

Municipal Planning Consultants (1981). "South Nation River Basin Development Study: Residential, Commercial, Industrial Component", report to South Nation River Conservation Authority.

O'Connor, D.J. and W.E. Dobbins (1958). "Mechanism of Reaeration in Natural Streams", *Transactions of the American Society of Civil Engineers*, 123, p. 655.

Owens, M., R.W. Edwards and J.W. Gibbs (1964). "Some Reaeration Studies in Streams", *International J. of Air and Water Pollution*, 8, pp. 469-486.

Palmateer, G. (1991). "Area of Impact of Rural Pollution Sources on the Water Quality of Surface and Sub-Surface Drainage in Southwestern Ontario", *Proceedings of the 23rd Drainage Engineers' Conference*, University of Guelph, Guelph, Ontario, pp. 69-95.

Peters, M.S. and K.D. Timmerhaus (1980). *Plant Design and Economics for Chemical Engineers*, 3rd ed., McGraw-Hill, Toronto.

Proctor & Redfern Limited (1984). "South Nation River Conservation Authority Floodline Mapping- South Nation River Final Report", South Nation River Conservation Authority.

Rowney, A.C. and C.R. MacRae (1989). "Current NPS Perceptions in Eastern Ontario: A Starting Point Survey", keynote address at Non-Point Pollution Workshop, Canadian Water Resources Association and the Soil Conservation Society, Carleton University, Ottawa.

Sawyer, C.N. and P.L. McCarty (1978). *Chemistry for Environmental Engineering*, 3rd ed., McGraw-Hill, Toronto.

SNRCA (1990), "Payne River Report: 1987-1989", Report, South Nation River Conservation Authority, Berwick, Ont.

Snoeyink, V.L. and D. Jenkins (1980). *Water Chemistry*, J. Wiley & Sons, Ltd., Toronto.

Tetra Tech, Inc. (1978). *Rates, Constants, and Kinetics Formulations in Surface Water Quality Modelling*, Environmental Research Lab, US Environmental Protection Agency, Athens, GA.

Thomann, R.V. and J.A. Mueller (1987). *Principles of Surface Water Quality Modeling and Control*, Harper & Row, San Francisco, CA.

US Army (1979). *HEC-2: Water Surface Profiles*, US Corps of Engineers, 609 2nd St., Davis, CA 95616.

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